THE NEW IRRIGATION ERA

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VOLUME III
NUMBER 4

DECEMBER '52
TO
AUGUST 1953

FOR DEPARTMENTAL CIRCULATION ONLY.

OUR FRONT COVER.

Sluice gate manufacture is an art in itself. In the past, the sluice gates were usually imported from abroad on account of their ready availability and competitive costs Even then some of the gates used to be manufactured indigenously The vast needs of the Post-Independence Era with its development projects particularly in the Andhra Desa of this State in the Godavari and Krishna Valleys and the difficulties of foreign imports made it imperative to install an up-to-date gate manufacturing plant.

The Tungabhadra Project was then under execution and a large number of other projects on the Krishna, Godavari and Pennar were in various stages of inauguration. These required a large number of flood gates, Tungabhadra Project alone requiring as many as thirty three gates of sixty feet span and twenty feet height, besides emergency gates for the penstock pipes, and a number of sluice gates for high and low level sluices and shutters for canal sluices in addition to penstock pipes. As immediate want was at Tungabhadra, it was decided to locate the plant for the manufacture of the gates at the Tungabhadra Headworks itself for the duration.

The front cover of this issue shows our Prime Minister, Shri Jawaharlal Nehru from an unusual angle, as he tacked the first weld during the inauguration of the Factory on 27th September, 1952. Elsewhere in this issue, an article on the salient features and achievements of this factory and a pictorial supplement of the various activities of the Plant have been included.

CONTRIBUTIONS INVITED.

Attention of the readers is invited to Circular No. 392/51-195 dated 26-11-1952 inviting articles and papers for publication in the New Irrigation Era. The Chief Engineer regrets that in spite of appeals and invitations, the contributions to the journal by the Officers of the Department and others have been much below expectations. The Officers of the Department, Principals of the Engineering Colleges, Officers of the Sister Engineering Departments in the State and others interested in the Science and Development of Irrigation are requested to contribute articles and papers in very good numbers.

For 'Instructions to Contributors' reference is invited to pages 245, 246, 247, 248 of Volume II. Copies may be had on writing to the Editor, Post Box No. 416, Chepauk, Triplicane Madras-5.

THE NEW IRRIGATION ERA

Published by the Irrigation Branch of the Madras Public Works Department, Madras (5), South India.

(For Departmental Circulation only)

" I believe in Hands that work,
Brains that think and
Hearts that love."

—Elbert Hubbard.

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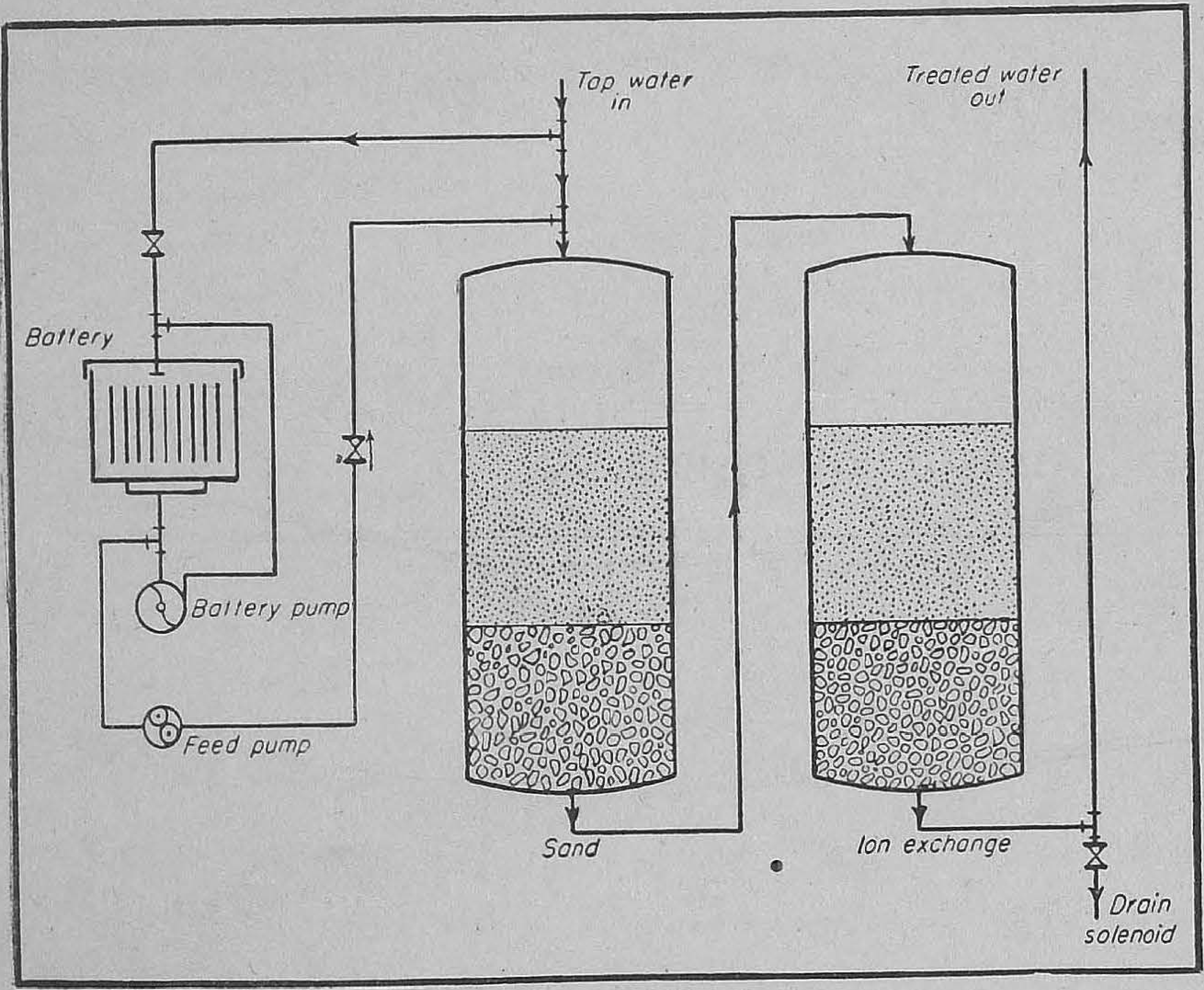
IMPROVED WATER TREATMENT

.... Means better water for better soft drinks with small-bubble carbonation, better flavour, and longer shelf life. Other food uses are expected for this new electrochemical water-purification treatment.

Water Treatment System Depends on...........



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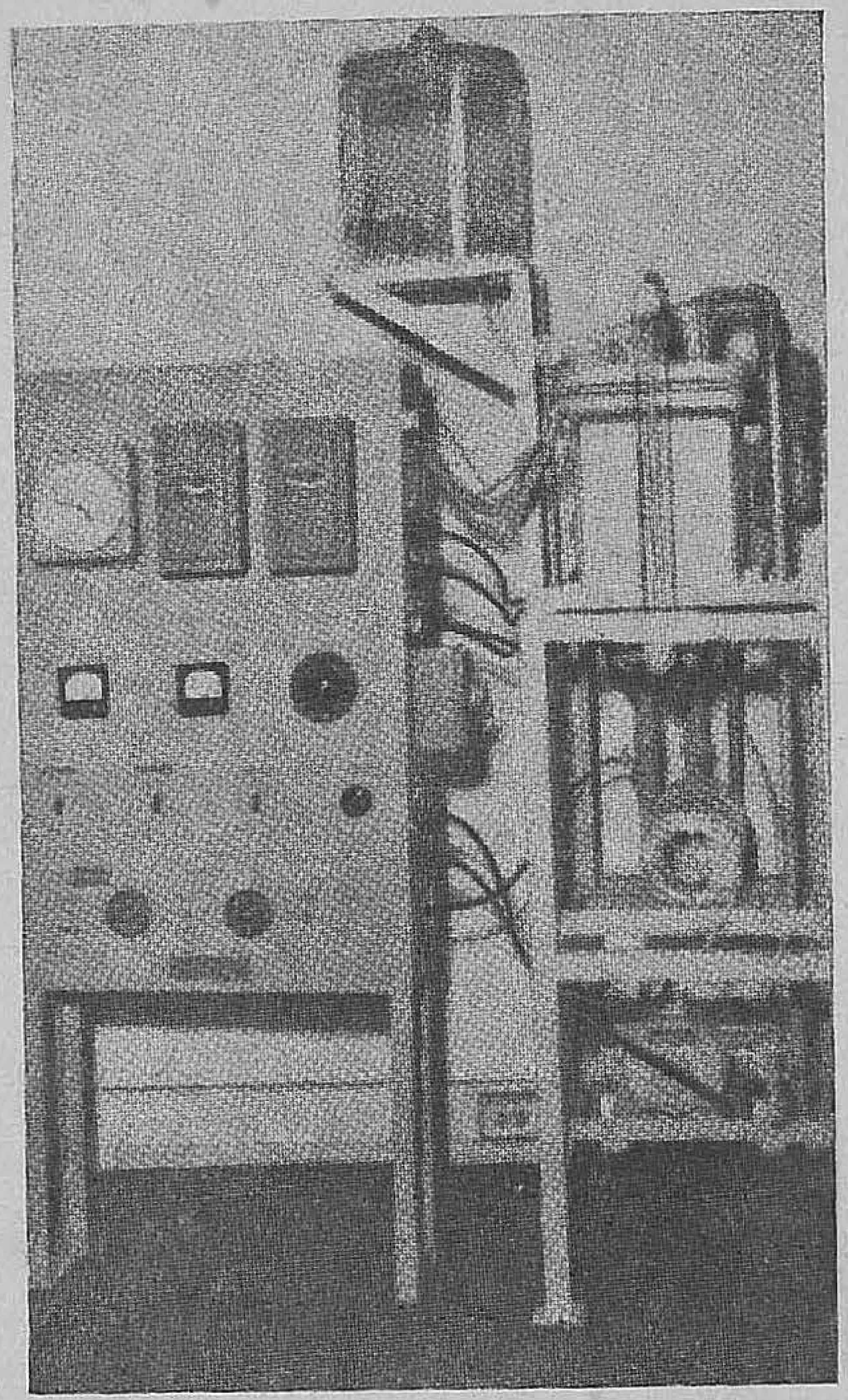


Several Soft-Drink bottling plants in the U. S. are getting better water with a new method of electrochemical water treatment. It is:

Decreasing foaming at the filler and so increasing the bottling rate.

Saving up to 25 per cent in dry ice for carbon dioxide production.

Giving "small-bubble" carbonation, corresponding to a retarded release of the supersaturated gas; beverages retain carbonation better on standing and thus have improved palatability.

Improving flavour and shelf life as organic matter has been removed from the water.

Further uses of this new process, developed by the General Hydro Co., are now seen for treating city tap or well water for brewing and processing dairy products, fruits, vegetables, meat.

It might appear that tap water, good enough to drink should be good enough to use for processing beverages and foods. But this is not always true as some water, while potable, is consistently unsuitable for processing and therefore needs treatment.

In other cases, water quality varies because of break-throughs of the filter, microorganisms resistant to chlorination, and seasonal variations such as prolific algae in the summer.

With well water, the distribution system has a great effect on quality. Such water although pure at the well, may have a high bacterial count at the tap. Hardness and alkalinity too must be modified; alkalinity, for example, is undesirable as it neutralizes the free acid in soft drinks.

(Continued in page 31)

EDITORIAL

OWING to the flood of technical literature that we are now getting from other countries and also from the Engineering Research Laboratories in India, there has been an increasing tendency towards over-elaboration of design, sometimes even at the expense of economy of construction. It is often forgotten that a successful Engineer is one who is able to get a work which costs a layman Rs. 10 carried out with an expenditure of Rs. 9. If, on the other hand, the Engineer produces an elaborate design which would cost Rs. 11, the services of such a person would not be wanted.

RECENTLY, designs for a summer storage tank for a Municipal water works came up for examination. The storage tank was in two parts to serve two towns with a common bund in-between. The design for the common bund was made elaborate, taking into consideration the suitability of the bund against slips during rapid draw-down and the stability of the bund when the tank on one side is full and that on the other side empty, and so on. This elaborate design was clearly unnecessary for the purpose on hand. In making out such a design, it was forgotten that the extreme conditions taken into account would never be obtained and even if there were to be a breach in the common bund, all that could happen is that the tanks on either side would balance in a short time and the breach is not likely to widen. The factor of safety in the bund could therefore be very much smaller. The design was eventually made simple with considerable saving in cost.

SO, I wish to emphasise once again that in planning works, we should never lose sight of the cost of execution. The simpler the design, the cheaper would be the cost of construction.

T. S. Venkataraman

HOW WOULD YOU DO IT?

Some of the most fascinating chapters in the life and memory of an engineer are those which deal with the unusual and unexpected situations which almost got him down but from which he finally emerged the victor.

—H. J. Gilkey.

About 1,000 ft. of 30-in. riveted sheet-metal pipeline was laid on the southwest slope of a mountain to conduct water from a very large seven-stage centrifugal pump to the headgate of a high-line irrigation canal some 300 ft. above the river. In the period of a few weeks after the line was assembled and riveted but not yet connected to the pump, the pipeline " crawled " progressively downhill at the rate of 4 or 5 in. a day. When the time came to connect the ends of the line, the upper end had receded several feet from the intake of the canal, and the lower end had by-passed the junction point for the pump connection. The closures were finally made by cutting a piece several feet long off the bottom of the pipe and riveting it to the upper end. Why did the pipe thus "crawl" slowly down the mountain side?

Editor's Note:—The above problem, is the second of a series drawn from actual experience and designed to demonstrate how one engineer overcame one of those "bugs" with which every engineer is familiar. Such experiences are an important part of an engineer's post-collegiate education. Hoping that these problems will prove valuable to those who have never encountered them before and entertaining to those whose memories are thus drawn back to similar experiences of their own, the editor welcomes contributions to this column.

Solution to this problem, will be, found on page 23. Alternative solutions are invited for publication in the 'New Irrigation Era' with appreciation by the Editor.

(Adapted with modifications from 'CIVIL ENGINEERING', June '52.)

Penetrometer Tests for Subsurface Exploration.

By Sri T. P. KUTTIAMMU, B.E., A.M.I.E.,

Executive Engineer, Kistna-Pennar Division.

Synopsis.

This article describes the penetrometer equipment used for subsurface exploration, its operation, usefulness and limitations. Corroborative tests used for verifying penetrometer indications are mentioned.

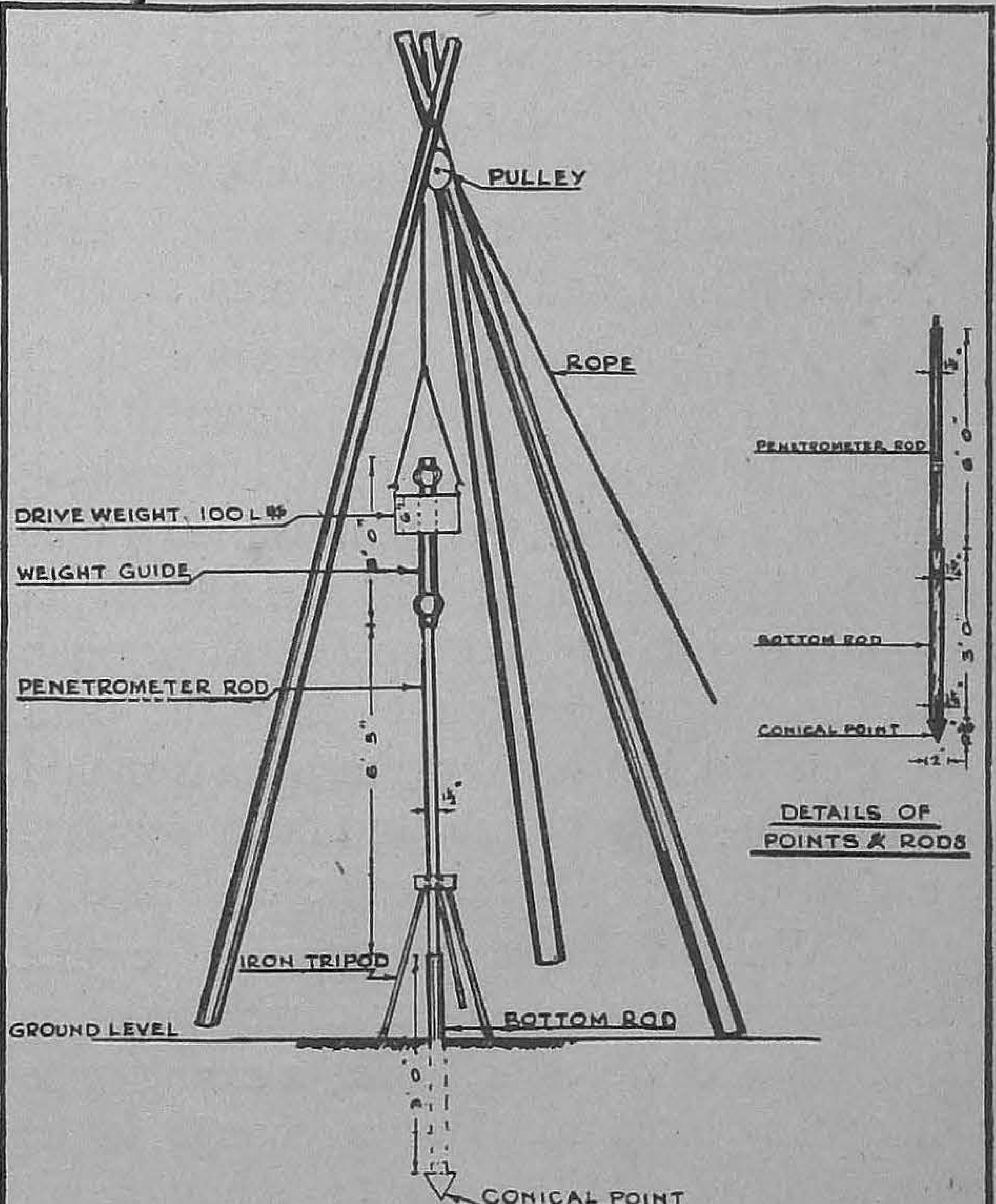
1. Introduction

1.1. Penetrometer tests are comparatively simple, cheap and speedy for subsurface explorations through loose soils. They give useful indications regarding the relative compaction of soils and subsoils and the levels of subsurface rock. These indications, however, should not be accepted without corroborative tests, as the penetrometer tests have serious limitations and may lead to erroneous conclusions if carelessly handled.

2. Equipment

2.1. The penetrometer is not a standardised apparatus. It is merely a probing rod with suitable arrangements for systematically driving it through soils until refusal and for extricating it afterwards. The equipment commonly used in the Madras Public Works Department is the one developed at Polavaram under the advice of Dr. Terzaghi in connection with the Ramapadasagar Project. Its component parts are shown in Fig. 1. They consist of three casurina posts 22 feet long and 6 in. dia, a conical shoe, a bottom rod, a few penetrometer (extension) rods, a 100 lb. drive weight with guide, a 10 in. dia. single shieve pulley, an iron tripod 5 ft. high, 1 in. dia. manilla rope, tommy bars and a few miscellaneous tools.

2.2. The conical point or shoe is a casting or is turned out of a 2 in. dia. steel rod. It is a solid cone 2 in. dia. and $2\frac{3}{4}$ in. long provided with a tenon at the base. The bottom rod is 3 feet long and has a slight taper, $1\frac{1}{2}$ in. dia. at bottom and $1\frac{3}{4}$ in. dia. at top. At the bottom is a mortice into which the tenon of the conical point can slip and at the top is a tenon with screw threads cut on it. The penetrometer (extension) rods are $1\frac{1}{2}$ in. dia. and 6 ft. long and are provided with a screw threaded tenon at one end and a corresponding mortice at the other. The 100 lb. weight is a cast iron piece capable of moving on a guide rod, which latter is again a $1\frac{1}{2}$ in. dia. rod having a screw threaded mortice at the bottom end. The movement of the weight on the guide rod is restricted to 30 in. by having two suitable shaped blocks welded on to the guide rod. Special care should be taken to ensure this welding to be perfect and strong, lest the blocks should come off as a result of the repeated knocks they receive while driving the penetrometer. The iron tripod has a circular ring and angle iron legs and is used to hold the penetrometer rods erect as shown in the figure. The casurina posts, pulley and ropes form the tripod derrick



PENETROMETER SETUP
(NOT TO SCALE)

Figure 1.

used to hold the weight and control its movement, downward for driving or upward for extracting.

3. Operation

3.1. The operation of a penetrometer test is simple. The casurina tripod with pulley and drive weight is erected over the point to be probed. The bottom rod with the conical shoe is kept on the ground and driven down slightly. A penetrometer rod is screwed on to the bottom rod at the bottom and to the guide rod at top held in position by the angle iron tripod, and hammered down by the 100 lb. weight dropping through 30 in. When this rod has gone down 3 or 4 ft., the guide rod is disconnected and another extension rod is introduced. In this manner probing is continued until the penetrometer strikes a hard surface and refuses to penetrate further. Chalk marks indicating foot lengths are made on the penetrometer rods and a maistry keeps a record of the number of blows taken for every foot ot penetration. The variation in the number of blows indicates the relative compaction of the soil and the refusal to penetrate indicates the presence of a hard surface.

3.2. After the penetrometer has been driven down to the required depth, it is withdrawn by reversing the hammering *i.e.*, by striking the guide rod upwards. The conical shoe will be left behind and cannot be recovered.

3.3. When probing is to be done in a slushy area or under water, special arrangements are necessary such as forming a sand island or using a steel punt. When a steel punt is used a hole is made in the centre of the punt and a 10 in. dia pipe inserted into it and welded water tight. Special care should be taken to have the punt firmly anchored, lest any movement of it should throw the rods out of plumb and make penetration and withdrawal difficult and complicated.

3.4. Where simple hammering cannot effect easy penetration, other devices, such as jetting from a compressor, using a casing pipe etc. may be resorted to. But a change in the method of driving will make the study of soil compaction from the penetrometer tests complicated.

3.5. For the recovery of the rods, screw jacks or differential pulleys can be employed, if reverse hammering is not successful. In the course of withdrawal, sometimes a few rods get lost by the threads getting worn out and loose or by breakage. If the breakage occurs near enough to the ground, a pit should be excavated and a large bottomless drum or a brush wood well sunk encasing the rod. The rod may then be connected to a penetrometer rod by using chain wrenches and tommy rods and the whole assembly hammered up and salvaged. If the breakage should occur at depths not easily approached, the cost of recovery would far exceed the cost of the rods and the rods should be abandoned. Such losses may be as much as 5 per cent but can be reduced by careful preservation of screw threads and handling of the operations.

4. Uses: Subsurface Rock

4.1. One of the main uses to which penetrometer tests are put is to get a contour survey of the subsurface rock underneath a sandy river bed or similar loose overburden. The area to be surveyed is demarcated by a system of co-ordinates referred to a base line, such as the axis of the river or the dam alignment or even longitudes and latitudes, and the points on the co-ordinates are systematically probed. To begin with points at large intervals, say 200 ft. or 400 ft. apart, along and across the axis are probed. Where more accurate data is desired probings are again taken at closer intervals. If wide variations are noticed in the levels of subsurface rock at a particular place the probings are taken at very close intervals, say 25 ft. apart. All points of doubt are tested out and cleared by repeated probing. The rock levels obtained in this manner are carefully plotted and the rock contours marked. Fig. 2 shows bed rock contours of the Pennar bed at Someswaram as traced by penetrometer probing. The river bed (sandy) at the site is about +225.0. Contours above +225.0 are surface rock and those below are subsurface rock.

4.2. Penetrometer indications in this respect however can be deceptive and should not be accepted without further corroboration. Presence of large size boulders or thickly packed shingle and gravel may resist penetration and should not be mistaken for rock. Verification can be done by hand-boring, calyx drilling or by geophysical survey. A few hand boring tests

spread evenly over the area covered by a large number of penetrometer probings will indicate the correctness or otherwise of the probings and will be useful as a check. Calyx or diamond drill borings to test the nature of the bed rock and to yield sample cores may be put in at select points and will give reliable confirmations. Subsurface rock contours obtained by penetrometer probings and independently corroborated by these borings, can be accepted as fairly accurate. Where the area concerned is large, a geophysical survey will also be useful. This survey by its very nature will however indicate only an averaged or generalised level of the hard surface, whereas the probings will indicate rock levels at specific, localised, isolated points. Discrepancies as high as 10 ft. may be seen to occur between the two sets of contours and will have to be correctly interpreted and got confirmed by core drilling.

5. Soil Compaction and Bearing Power

5.1. A penetrometer test can also yield useful data regarding some of the characteristics of the soils probed. This is studied from the penetrometer graph. Please see Fig. 3 wherein a few typical graphs are given. The curves indicate the number of blows imparted for each foot of penetration and give a comparative idea of the relative compaction, bearing power and other simple characteristics of the soil at various depths in a general way. Where the penetration is easy, the soil is loose and where it is very difficult, the soil is hard. In between we may get soils of varying degrees of compaction. Where the curve is erratic, indicating high resistance followed by easy penetration in repeated succession, presence of pebbles and stones may be suspected. If the top soil in a river bed is seen to be loose for 20 or 30 ft. uniformly over a large area the same may be considered liable to scour

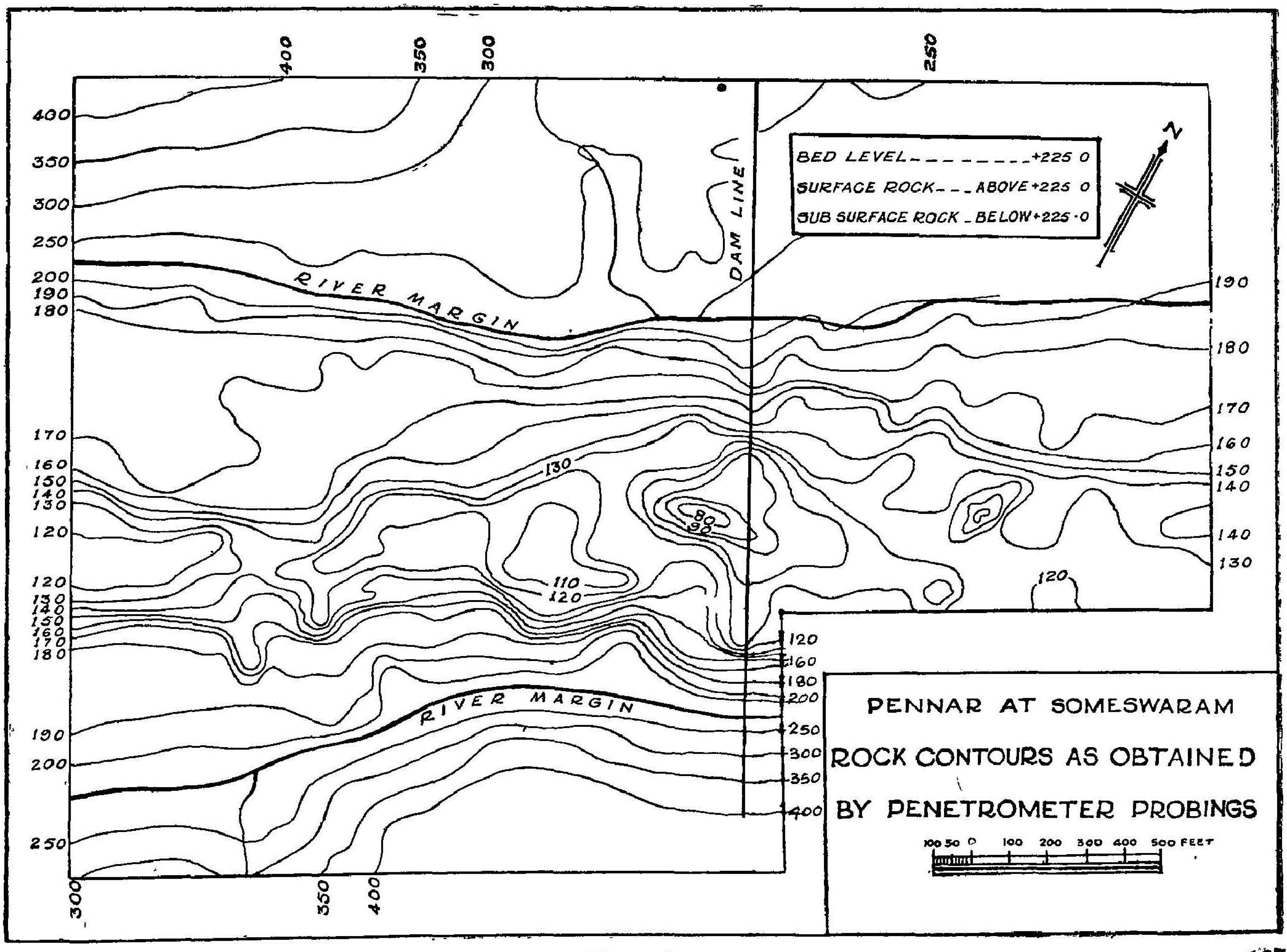
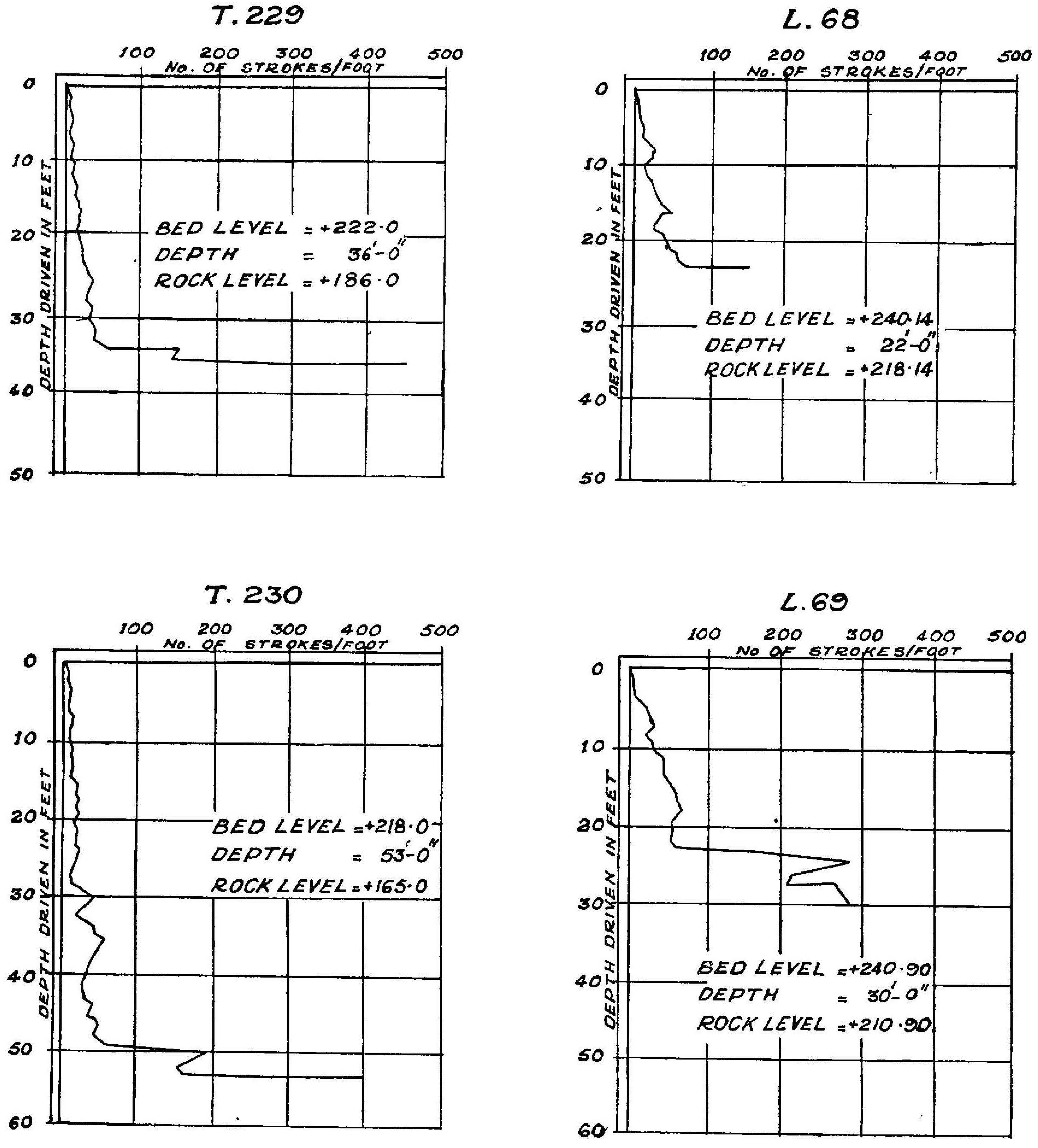


Figure 2



PENETROMETER GRAPHS

TYPICAL SAMPLES

Figure 8.

during high floods. Made up soils in the foundations of a large building may also be located this way.

5.2. Here again there are serious limitations to be borne in mind. The penetrometer is not a standard apparatus and the graphs obtained from them connecting depth of penetration to number of blows give only comparative values. With increasing depth of probing frictional resistance also increases (though the conical shoe and the bottom rod are intentionally made slightly larger than the extension rods) and the number of blows required for every foot of penetration need not necessarily indicate the compaction or bearing power of the soil. Corroborative tests should therefore be made wherever necessary.

5.3. Soil samples taken by hand boring at a few places can be analysed and tested for various soil contents. Undisturbed samples may be taken in a few places and tested for compression, shear etc. Actual loading tests may be made to determine bearing power. Or better than all these a few standard penetration tests can be made. This test is described in Dr. Terzaghi's "Soil Mechanics in Engineering Practice—Chapter VII. Pages 265-266". A few standard penetration tests in combination with a large number of simple penetrometer probings will give useful and reliable data on soil compaction and bearing power over the entire area covered by them.

6. Cost Data

6.1. A penetrometer set with rods sufficient for a 200 ft. probe costs about Rs. 850 at the present market rates. For operating this set, a gang of 6 to 8 mazdoors and a literate head mazdoor are ordinarily necessary. The cost of driving through sand, sand mixed with stones or loose loamy soils will be about Rs. 0–6–6 per foot for the first 70 ft. Rs. 0-10-0 per foot for the next 30 ft. and Re. 1-0 0 per foot for depths exceeding 100 ft. Extraction will cost about Rs. 0-7-0 per foot average. A conical shoe costs Rs. 6–0–0 and for every probe a shoe has to be abandoned The cost of a penetrometer rod is Rs. 15-0-0 for 6 ft. length and allowance should be made towards losses upto five per cent—vide para 3.5. above. Repairs to drive weight and guide, renewal of casurina posts and ropes etc. are other incidental expenses. Inclusive of all these the cost of a penetrometer test may be taken roughly to vary from Rs. 1-0-0 to Rs. 1-8-0 per foot depth of probing depending on the nature and depth of soil. A hand boring test under identical conditions may take more time, say about four times more and cost from Rs. 2-0-0 to Rs. 3-0-0 per foot.

7. Conclusion

7.1. Where extensive exploration of soil and subsoil characteristics and subsurface rock contours has to be done, it is simple, quick and economic to go in for penetrometer tests. Probings should be done extensively over the whole area to be studied and intensively at places of importance or of doubt. Indications given by the tests should be verified by a few corroborative tests. Hand boring, calyx drilling and standard penetration tests can generally give all data required for such verification.

Gaugings in Rivers and Channels.

(Computations of Discharges from Limited Observations)

BY SRI N. S. ANANTHANARAYANAN, B.E., A.M.I.E.,

Executive Engineer, Gaugings Division.

1. Introduction.

1.1. Gaugings in Rivers are conducted to assess the probable discharges against all stages of flow. The problem of gaugings cannot be brushed aside lightly, as it forms the vital link between the availability of water and the proper distribution thereof. This is a problem of husbanding the hydraulic resources to the maximum advantage which is one of the main functions of an Irrigation Engineer.

1.2. An important practical application of gaugings is to find out how to send down varying discharges through a regulator for the ayacut of the river or channel taking off from it at different times of the cultivation season. The obvious method is to work out the discharge with reference to vent areas and heads of discharge by applying suitable co-efficients of discharge. Such a system might be labouring under the drawback of actual realisations being at variance with the anticipated values from empirical considerations. A more representative system would therefore be called for, atleast until such a time as the water course attains its regime which will permit of a co-efficient being fixed on a permanent basis.

1.3. For this purpose a number of methods of gauging the flows are resorted to and one of the standard methods is to assess the velocity of flow by means of a Current Meter and calculate the discharge by the standard formula $Q = AV$; where Q is the discharge in cusecs, A is the area of section in sq. ft., and V the velocity in ft/sec. The sectional area required for the pur-

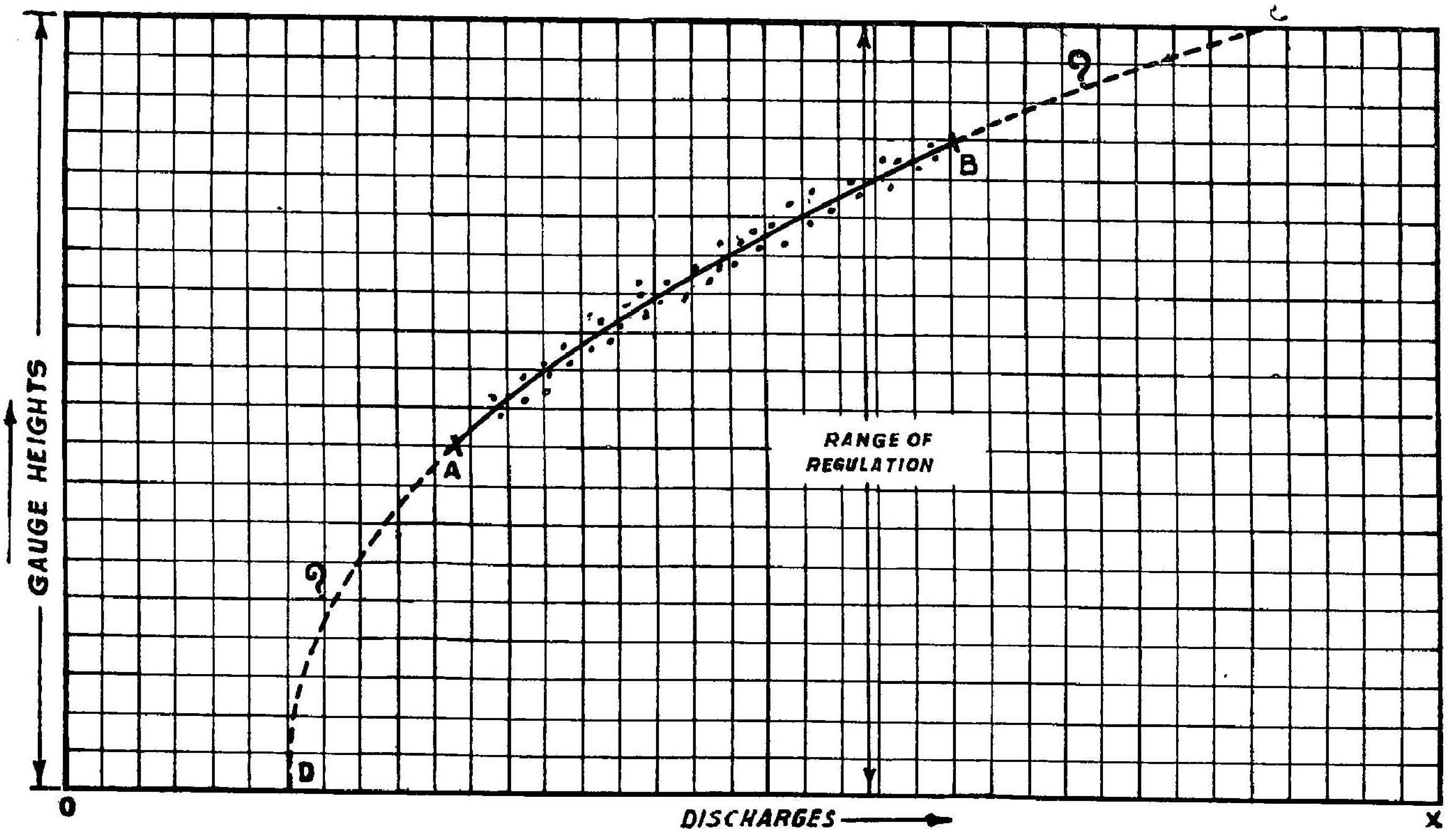


Fig. 1 :—Computing discharges beyond range of observations by approximate extension of Curve.

Legend.

AB—Range of Observations

AD—Approximate extension of Curve with a dip towards the X axis

BC - Approximate extension of Curve.

8 MADRAS

pose will be determined side by side with the velocity observations. These discharges are worked out for different stages of flow which are recorded with reference to permanent gauges fixed at the site as well as at certain cardinal points in their vicinity, say for example, the rear of Regulators from which there are offtakes wherein the gauging operations are conducted. A curve of Discharges against Gauge Readings is plotted and kept for guidance in regulating the supplies during the irrigation season. For a required discharge to be passed down the course, the gauge reading is read from the graph and the regulator shutter openings are manipulated for this reading.

1.4. For the Discharge-Gauge Reading Curve to be absolutely representative, Current Meter observations have to be conducted regularly throughout the irrigation season and should cover every stage from "Low to Normal" and "Normal to High" with a fair number of observation points for every one of these conditions. Ideals like this are rarely realised in actual practice in spite of the best of efforts and often times the computer of the Discharge-Gauge reading curve is faced with the intriguing problem of forecasting the probable discharges at the low and high stages from a well drawn and representative curve in the mid ranges and possibly some distance either way from it (Figure No. 1).

2. Standard Methods.

2. This article deals with two of the standard methods adopted for computation of discharges from limited observations. These are :

1. $A\sqrt{D}$ (Stevens) Method.
2. Running Method.

A brief description of these methods will be given and some typical cases will also be examined using these systems.

3. Stevens' Method.

3.1. The first method has been evolved by Mr. J. C. Stevens and described in his paper "A method of estimating stream discharges from a limited number of gaugings" published in the Engineering News Record of July 18th, 1907.

3.2 The genesis for this method is got from Manning's equation.

$$\left(V = \frac{1.486}{n} r^{2/3} S^{1/2}\right)$$

with the standard notations—*viz.*,

V = Velocity ; n = Co-efficient of rugosity,
r = Hydraulic mean radius, and
s = Slope.

In a representative metering section the water level and the standard Gauge reading must register truly and the cross sectional area must be uniform for all practical purposes. These two factors can be accurately assessed by elementary Levelling and Sounding and the mean velocity curve drawn, from which interpolation for lower and higher stages would be possible, by extension.

3.3. On a fair approximation, specially for the higher stages of flow $\frac{s^{1/2}}{n}$ may be taken as constant and so V can be written in the form, $V = Kr^{2/3}$; all other factors in the Manning's equation except "r" being constants or assumed to be very nearly so. The value of K can be determined from the available range of gaugings from which the mean velocity curve has been plotted, as the value of 'r' can be worked out for every stage. A curve combining K with the gauge height can be extended for other stages and the value of K obtained therefrom can be multiplied by $r^{2/3}$ and 'A', which will furnish the discharges in the speculative reaches.

3.4. The Stevens' method is a further simplification of the above system in which $D^{1/2}$ or $D^{2/3}$ is substituted for $r^{2/3}$ used in the routine formulae. "D" here is the mean depth against the hydraulic mean radius "r" *i.e.*,

$$D = \frac{\text{Cross sectional area}}{\text{Width of water course at the water level}}$$

$$r = \frac{\text{Cross sectional area}}{\text{Wetted perimeter}}$$

The use of 'D' against "r", which latter is the more accurate factor can be tolerated where the width of water course would approximate to the wetted perimeter. This will be the case only if the water course is very wide and is comparatively shallow. Here the width at water level can be taken as equivalent to base width plus side slopes. For these conditions

the Chezy's formula for discharge in open channels *viz.*;

$Q = CA\sqrt{r}.\sqrt{s}$ where "C" is a contant; 'A' the area of cross section; 'r' the hydraulic mean radius and 's' the slope, can be written in the form $Q = CA\sqrt{D}\sqrt{S}$.

As in the previous case $C\sqrt{S}$ may be assumed to be a constant in the higher stages and Q can be taken to be a function of $A\sqrt{D}$.

3.5. As both A and D are connected with the gauge height, the values of $A\sqrt{D}$ can be calculated for all stages and plotted. In the same curve, the discharges as observed in the limited range can be plotted against $A\sqrt{D}$. This curve can be extended upwards and such an extension would be a straight line when $C\sqrt{S}$ is practically a constant which will be so under conditions of high stage flows. From these two graphs discharges corresponding to any gauge height can be interpolated as shown in Figure 2.

3.6. By using Manning formula in Stevens' method, the exponent for D should be two-third instead of one-half and some times correlation from curves of $AD^{2/3}$ versus Gauge Height and Q versus $AD^{2/3}$ gives more realistic results than those for $AD^{1/2}$.

4. Running Method

4.1. This method is eminently useful in fixing discharges at the lower stages and the assumption made is that the Discharge–Gauge curve in this zone is a parabola whose vertex corresponds to a gauge reading for zero discharge. This reading is purely hypothetical and practical values of discharge would result in stages higher than it. However, to determine the shape of the parabola, the vertex is a

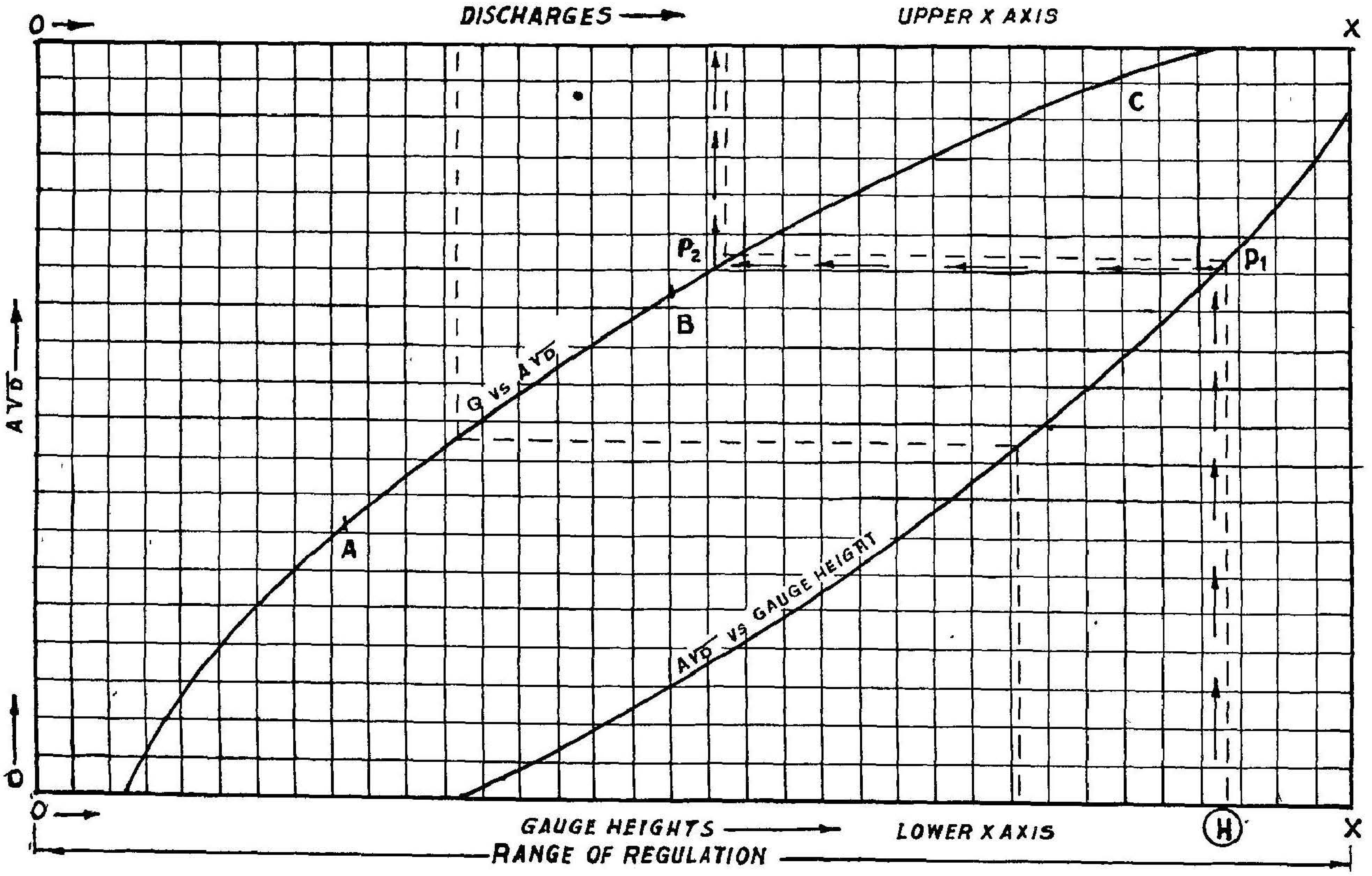


Fig. 2:—Stevens' Method of computing discharges.

Legend.

AB—Range of observations, BC—Extension of curve.

To assess the discharge for a gauge height (H) beyond the range of observations follow the line of the arrows from H on the lower X axis to the point of intersection P1 on the $A\sqrt{D}$ versus Gauge Height Curve and thence to the point of intersection P2 on the Q versus $A\sqrt{D}$ Curve and finally 'Q' on the upper X axis.

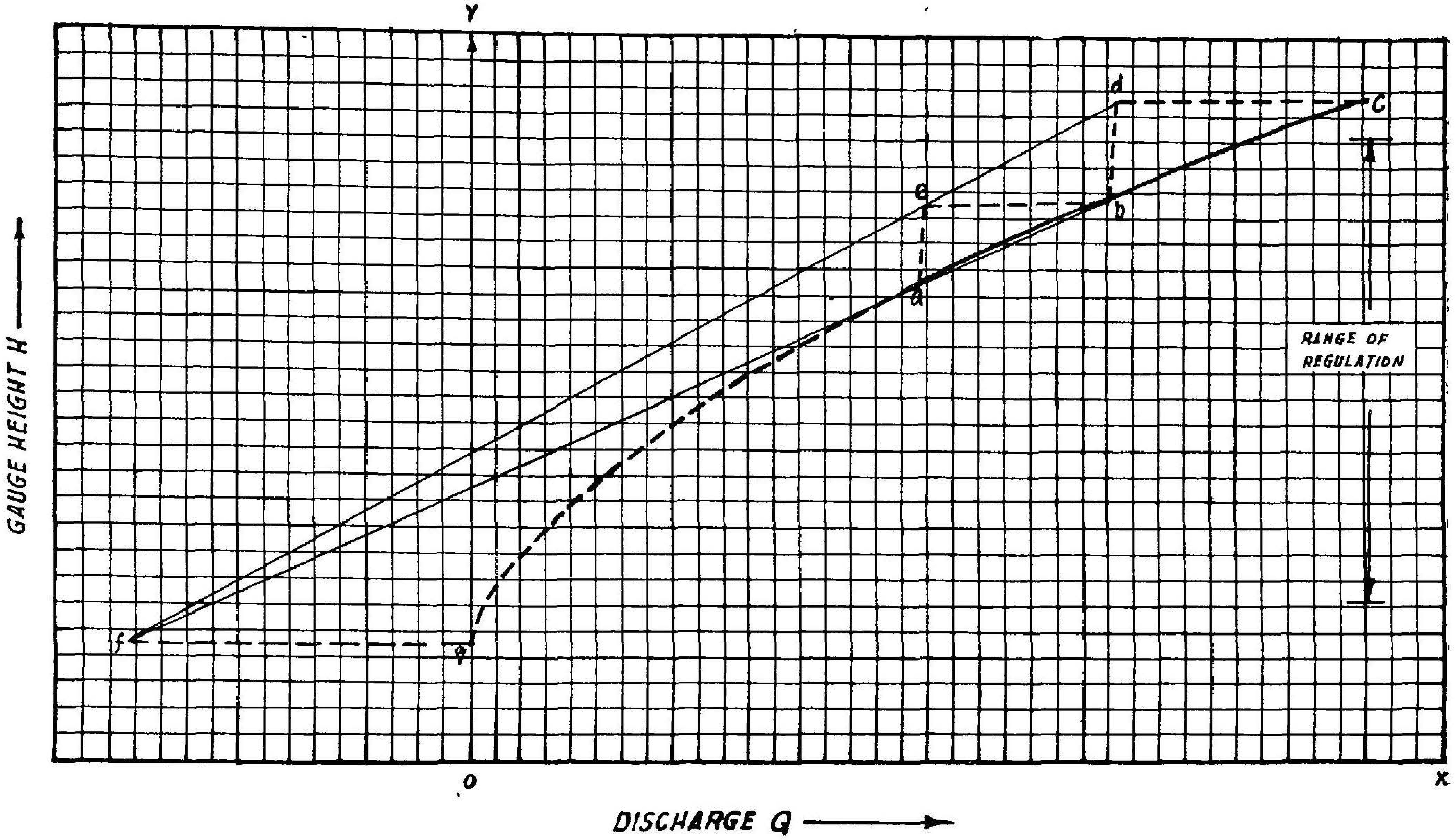


Fig. 3 : – Dr. Running's Method.
Legend.
ac – Range of observations
The abscissae of a, b, c, any three points on the curve must be in geometrical progression. V, is the vertex of the parabola passing through a, b, c. Va is the probable curve for discharge in low stage.

necessary factor and the construction for this is given by Dr. T. R. Running, Professor Emeritus of Mathematics, University of Michigan and is shown in Figure 3.

4.2. Three points in the gauged range a, b, c, are so chosen that their abscissae are in Geometric progression (r, r^2, r^3) and verticals through a and b are drawn to meet the horizontals through b and c at e and d. The line joining d, e and the chord of the assumed parabola "b a" produced backwards meet at f. The projection of 'f' on the vertical through zero discharge gives the vertex V of the parabola. A parabolic curve connecting V and a, b, c, defines the probable discharge at the lower stages. The mathematical proof for the construction is furnished in the appendix to this article.

5. Study of Methods

5.1. The methods recommended above are made use of in two typical cases in order to study their scope and draw inferences therefrom. The cases examined are, those of the gaugings conducted in the Tanjore Delta during 1952–53 in,

(A) River Cauvery below the Grand Anicut.

(B) Grand Anicut Canal taking off from the Grand Anicut.

The Graphs for "Discharge-Gauge Reading" are drawn for these two cases.

5.2. Two curves, one by the approximate method of extending the general curvature of mid reach to the upper reach and giving a smooth dip from the mid reach to the lower reach and the other curve showing probable discharges by the Stevens' Method in the upper reach and Running's Method in the lower reach are plotted for case A (Figure 4). A separate graph for obtaining discharge by Stevens' Method for use in Figure 4 has also been drawn in Figure 5. For case B, however, Stevens' Method is omitted as the number of observations in the upper reaches are quite

RIVER CAUVERY BELOW GRAND ANICUT

GAUGE HEIGHT—DISCHARGE CURVE

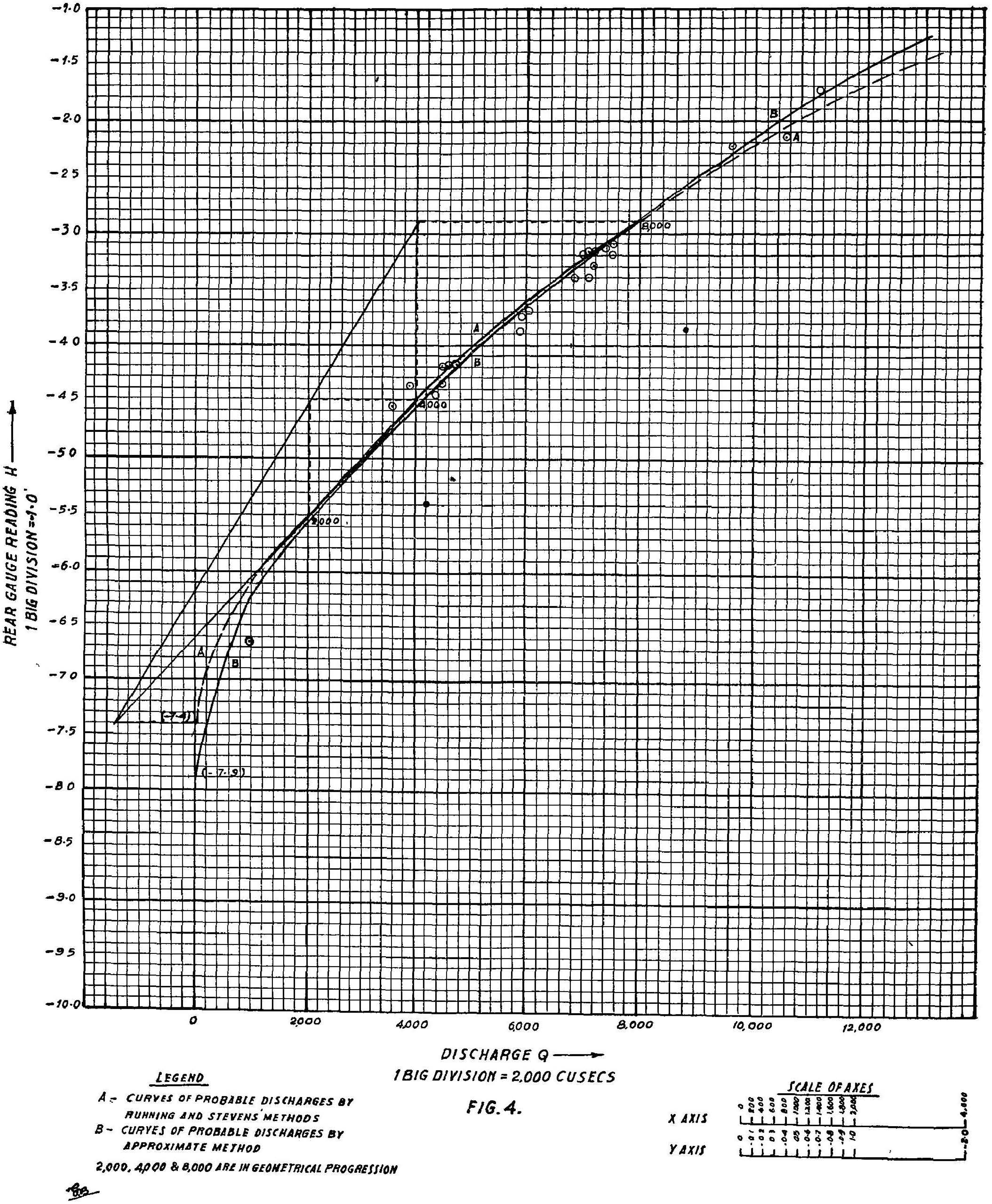


FIG. 4.

RIVER CAUVERY BELOW GRAND ANICUT-DISCHARGES BY STEVENS' METHOD

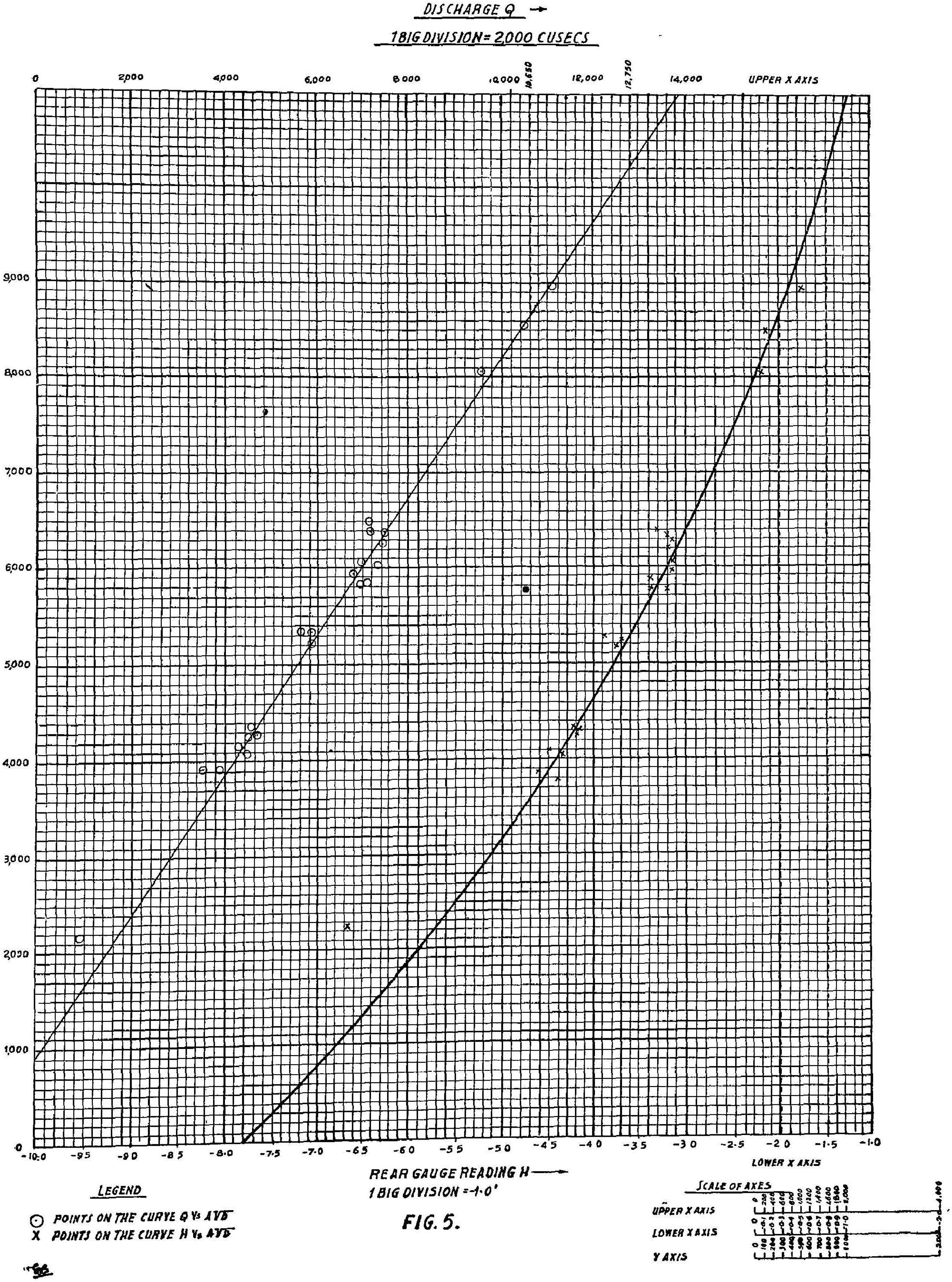


FIG. 5.

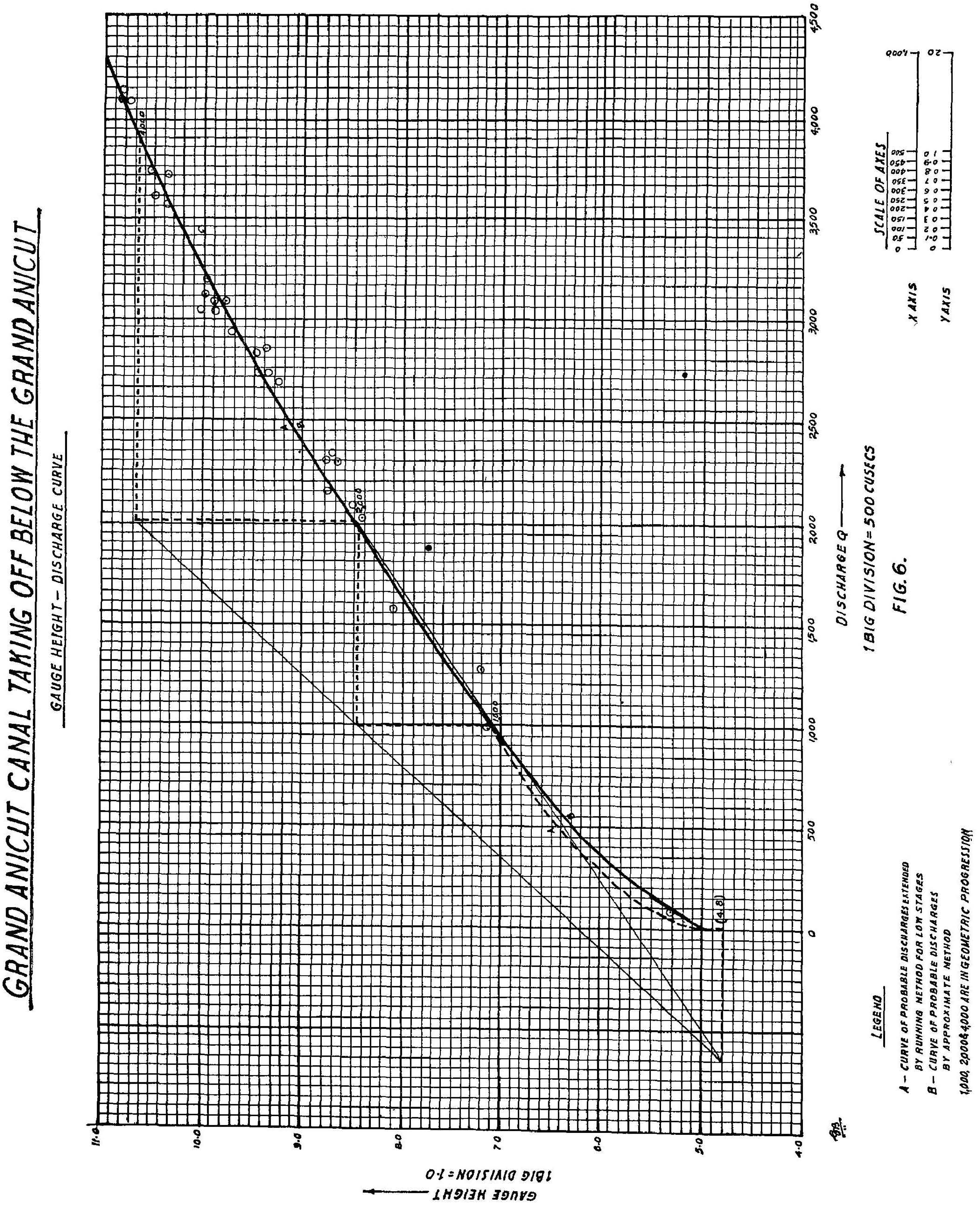

GRAND ANICUT CANAL TAKING OFF BELOW THE GRAND ANICUT
GAUGE HEIGHT - DISCHARGE CURVE
0
500
1,000
1500
2000
2500
3,000
3500
4,000
4,500
DISCHARGE Q
1 BIG DIVISION = 500 CUSECS
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
GAUGE HEIGHT
1 BIG DIVISION = 1.0
(4.8)
SCALE OF AXES
X AXIS
Y AXIS
LEGEND
A - CURVE OF PROBABLE DISCHARGES EXTENDED BY RUNNING METHOD FOR LOW STAGES
B - CURVE OF PROBABLE DISCHARGES BY APPROXIMATE METHOD
1,000, 2,000 & 4,000 ARE IN GEOMETRIC PROGRESSION

FIG. 6.

representative. Apart from this, the Grand Anicut Canal is a well defined water course of comparatively recent origin and the conditions of a large bed width and shallow depth which are the basic essentials for the Stevens' method cannot be applied here. So in this case only the Running Method is adopted for the lower reaches (Figure 6) and the curve by the approximate method is also drawn in this graph for comparative study.

5.3. The results of examination are furnished below in tabular statements :—

TABLE I

River Cauvery Below Grand Anicut—Comparative Statement of Discharges.

Serial No.	Gauge Reading H	Stage	Discharge in Cusecs			Percentage of Deviation from Approximate Method
			Approximate Method	Running Method	Stevens' Method	
1	—7 00	Low	500	160		— 68.00
2	—6 50		780	520		— 33.33
3	—6.00		1,320	1.120		— 15.15
4	—5 50		2,100	2,000		— 4.76
5	—5.00	Average	2,960	2,920		— 1.35
6	—4 50		*3,990	4,000		+ 0.25
7	—4 00		5,100	5,070		— 0.59
8	—3.50		6,310	6,200		— 1.74
9	—3.00		7,620	7,600		— 0 26
10	—2 50		8,960		9,000	— 0.45
11	—2.00	High	10,400		10,650	+ 2.40
12	—1 50		12,000		12,750	+ 6.25

* Slight deviation due to minor changes in Discharge curves by both methods

TABLE II

Grand Anicut Canal Taking off Below Grand Anicut—Comparative Statement of Discharges.

Serial No.	Gauge Reading H	Stage	Discharge in Cusecs.		Percentage of Deviation from Approximate Method
			Approximate Method	Running Method	
1	5.00	Low	...	...	...
2	5.50		145	90	— 37.90
3	6.00		350	280	— 20.00
4	6.50		620	550	— 11.29
5	7.00		940	930	— 1.06
6	7.50	High and Average	1,280		
7	8 00		1.650		
8	8.50		2 035		
9	9.00		2,430		
10	9 50		2,835		
11	10.00		3,280		
12	10 50		3,800		
13	11.00		4,380		

6. Inferences

6.1. The following observations and inferences can be drawn from this study :—

In the case of the River Cauvery, the gauge reading corresponding to zero discharge which is—7.4 by the Running Method agrees fairly with —7.9 by the Approximate Method for the same condition. In the case of the Grand Anicut Canal the gauge reading for zero discharge is 4.8 by the Running Method against 4.9 by the Approximate Method.

6.2. On the other hand, the discharges in the lower stages by the Running Method are substantially lower than those by the approximate method in both the cases; the variation being in the range of about 15 per cent to 70 per cent for River Cauvery and 10 to 40 per cent for the Grand Anicut Canal. Once the mean discharges range is reached there is practically no difference. In the higher ranges, the fluctuation between the Stevens' method and approximate method is only about four per cent on an average, the sign of variation being positive in this reach against negative in the lower regions.

6.3. These show that whilst the principle of dipping the curve in the lower ranges defines the shape of the curve, arbitrary methods in drawing this curve give exaggerated values for discharge. In the case of the higher reaches though Stevens' method does not show large variations, it would be more technical and possibly realistic too, to adopt this method in preference to the upward extension of the curve in the average reaches in a haphazard manner.

6.4. The views of Mr. G. Bertram Kershaw given in his book on " River Gauging " in this aspect, which follows the inferences drawn above makes an interesting study. It reads : " More discharge measurements are needed to establish the lower part of the station discharge curve than for any other portion, since small differences in gauge height have a much larger proportional effect upon the discharge for low stages than for high : the slope of the river too, changes faster in the low than in the higher parts of the curve. As a general rule, incomplete flow records during " low water " periods can be interpolated without material error ; but if this is done where "high-water" periods are concerned, serious errors may be introduced."

6.5. It would also be useful to study how far the errors observed in both ranges would affect economy in distribution of water. An error of—70 per cent in the low stages of flow would mean shorter supply of water and the savings so effected will be of the order of about 350 cusecs in 500 cusecs for a comparatively shorter duration than in the high stages. On the other hand a +4 percent error in the higher stages in a supply of about 12,000 cusecs would mean excess issue of 480 cusecs for a much longer time. Hence it would be seen that for judicious distribution of water, it is essential to have accurate ideas of discharges both in the low and high stages, specially the latter.

7. Conclusion

7.1. The above examination defines the broad outlines on which the rational methods of computing discharges can be made use of. There is yet another method known as " Logarithmic Method of Extension " in which the Discharge is expressed in terms of the Gauge height by making use of the formula,

$Q = C (G - a)^n$ where Q is the Discharge in cusecs, G is the Gauge height in feet a is the initial Gauge Height for zero discharge, and C and n are site constants. The use of this expression for computation involves the use of Logarithmically ruled paper and so the method is not elaborated any further, in this paper.

7.2. It will however, be obvious to those, who are entrusted with the responsible task of gauging water courses, that only two courses are open to them. Either they must carry out their observations in all the ranges in as systematic a manner as possible or they must do some serious guessing in regard to the non-observed regions with lots of graphs and tables. It seems better to choose the direct observation alternative which is more straightforward and representative of actuals rather than go a speculating, albeit in a rational and technical manner.

8. Acknowledgements

8. The methods discussed in this article are described in the book on " Hydrology " by Wisler and Brater. The author has also received valuable assistance from Sri K. Jayabalan, Junior Engineer of his office, in the compilation of data and plotting of the graphs.

APPENDIX

Proof for Dr. Running's Method:

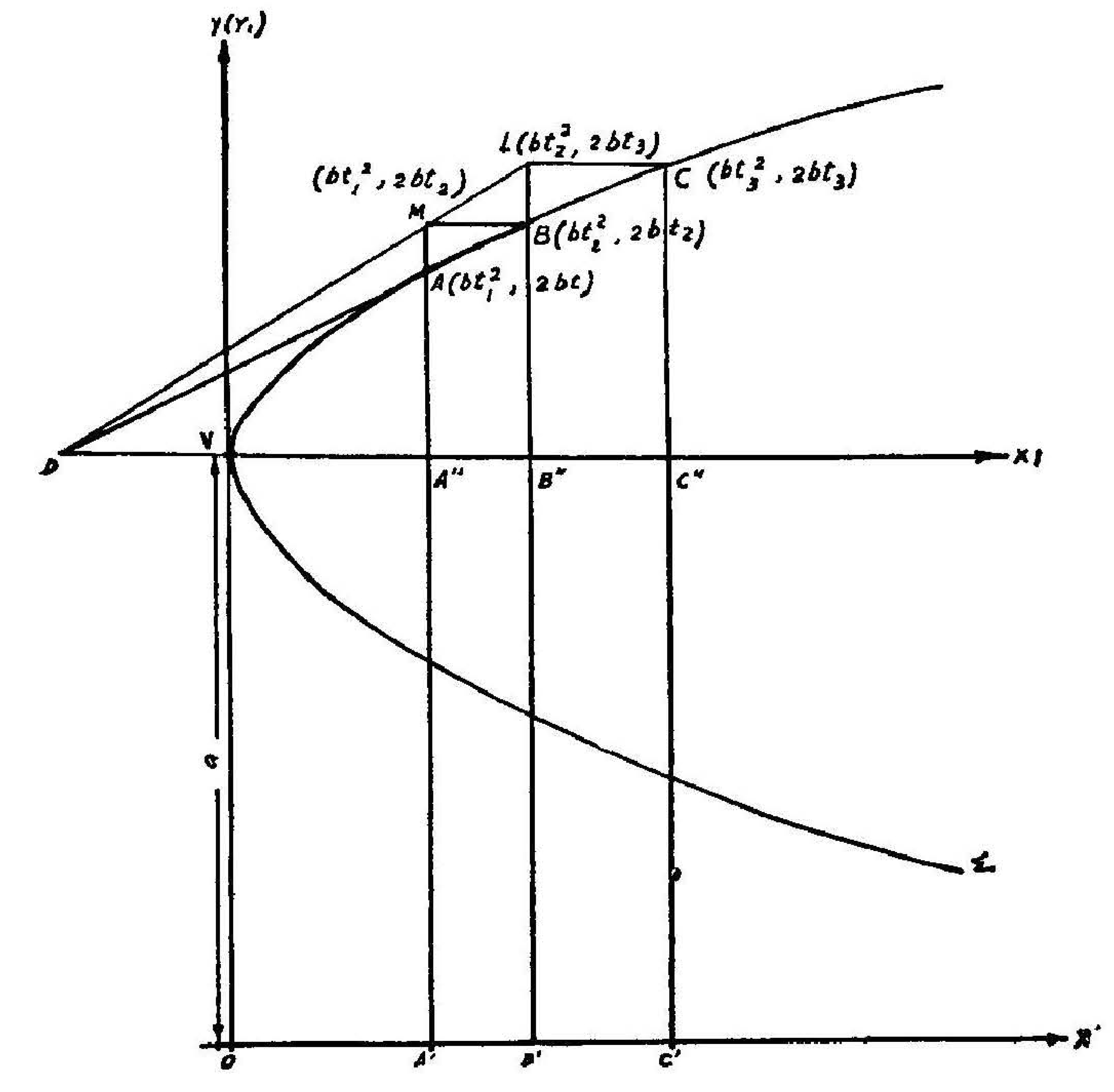


Fig. 7.

Let OX, OY be the given axes

Let ⋚ be the given Parabola

Let A, B, C be any three arbitrary points on it.

Let the horizontal at B meet the vertical through A at M and the horizontal at C meet the vertical through B at L.

Let BA and LM meet at D.

Through D draw a Horizontal D X_1 to meet the curve at V. Then assume V to be the vertex of the Parabola.

It has to be proved that unless A, B, C are points on the Parabola whose abscissae OA', OB', OC', form a Geometric Progression, DV will not be the axis of the Parabola.

Proof: Change the origin of O to V

Let Vx_1 and Vy_1 be the new system of axes.

If we suppose OV to be "a" and the Latus Rectum of the Parabola to be "4b", then the equation of the Parabola with respect to OX, OY is $(y-a)^2 = 4\,bx$. (Because $y^2 = 4$ b.x. is the equation of the same Parabola with respect to Vx_1 and Vy_1 as axes).

Let A, B, C be $(bt_r^2, 2bt_r)$ where r = 1, 2, 3 respectively.

Consider the similar triangles DLB'' and DMA''.

$$\frac{DB''}{DA''} = \frac{LB''}{MA''}$$

$$\therefore DA'' = \frac{DB''.MA''}{LB''} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Consider the similar triangles DBB'' and DAA''

$$\frac{DB''}{DA''} = \frac{BB''}{AA''}$$

$$\therefore DA'' = \frac{DB''.AA''}{BB''} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Equating (1) and (2)

$$\frac{DB''.MA''}{LB''} = \frac{DB''.AA''}{BB''}$$

$$\therefore \frac{BB''}{LB''} = \frac{AA''}{MA''}$$

Now BB'' = ordinate of B = 2 bt_2

AA'' = ordinate of A = 2 bt_1

LB'' = CC'' = ordinate of C = 2 bt_3

MA'' = BB'' = 2 bt_2 (as before)

∴ By substituting

$$\frac{2\,bt_2}{2\,bt_3} = \frac{2\,bt_1}{2\,bt_2}$$

$$\therefore (t_2)^2 = t_1 t_3$$

Squaring this equation and multiplying both sides by b^2,

$$b^2\, t_2^4 = (b\, t_1^2).(bt_3^2)$$

or $(bt_2^2)^2 = (bt_1^2).(bt_3^2)$

Again bt_2^2 = VB''

bt_1^2 = VA''

bt_3^2 = VC''

∴ $(VB'')^2$ = VA''.VC''

On transformation to original axes, we get,

$(OB')^2$ = OA'.OC' as there is no change in abscissae from V to O.

That is, the square of the abscissa on the middle point of A', B', C', viz., B' is equal to the rectangle contained by the abscissae of the other two points A' and C'

∴ A, B, C are not arbitrary points but are points whose abscissae form a G. P.

(Since if r, r^2, r^3 be in G. P. then $(r^2)^2 = (r).(r^3)$)

∵ The assumptions that DV is the axis of the Parabola and V is the vertex of the Parabola are true.

Manufacture of Gates and Shutters at Tungabhadra.

SRI G. SRIRANGACHARY, B.E., M.E. (HONS)., M.I.E ,
Executive Engineer, Workshops and Machinery Division.

Conserving of Steel by the use of up-to-date techniques of production is important for the economy of our Country. A beginning in this direction has been made at Tungabhadra when the Prime Minister of India, Pandit Jawaharlal Nehru, made the First Weld and inaugurated the Shutter Manufacturing Factory, the first of its kind in India, at the Tungabhadra Dam site on 27th September 1952.

The Shutter Manufacturing Factory has been set up at Tungabhadra for the speedy and economical manufacture of—

(i) Gates for regulating the surplus flow over the Dam ;

(ii) The Equipment required for operating the Gates;

(iii) Shutters required for controlling the Irrigation Sluice openings;

(iv) The devices which are fixed in the Dam for operation of the Gates and Shutters; and

(v) The Shutters and Operating gear for the Canal Systems.

Manufacture of Gates for the Over-flow of the Tungabhadra Dam :—The Tungabhadra Reservoir can be expected to get filled up in about a month after the outbreak of the South West Monsoon. From July till October, the surplus waters of the Tungabhadra River will surplus over the spillway section through its 33 openings, each 60 feet wide. There will thus be a three furlongs length of water rushing down to the foot of the Dam about 100 ft. below the crest of the spillway.

The 33 openings in the Spillway would require 33 Gates each 60 ft. in width and 20 ft. high, to control the onrush of floods, and later to help storing the maximum quantity of water over and above the top level of its crest.

The weight of each Gate including its operating platforms, guide, rails etc., is 80 tons. For the 33 gates the total weight of steel to be fabricated is over 2,600 Tons. The factory has been designed to manufacture all these items within a period of fifteen months.

A special feature of this Factory is the attainment of economy and speed—

(i) by adopting the Shop-Welded Field-Rivetted technique and

(ii) by the use of the 'Flow-Line' System.

Shop-Welded Field-Rivetted Method :—In the technique known as the "Shop-Welded Field-Rivetted" method the various pieces that go to make the structure are welded together to form 'Unit-Sections' that are convenient for handling from the Shop to the Field.

In the Field, the various sections which are manufactured in the Shop by welding are rivetted together into one whole structure.

An inherent difficulty of the welding technique experienced in advanced Countries where this method is now in vogue, resulted in some failures in the past owing to insufficient control of the welding. To ensure adequate control, experience has shown that the welding has to be done in the Shops equipped with up to-date apparatus to detect any weak spots without the necessity for costly tests to destruction.

Rivetting in the Field has to be resorted to in preference to welding for the simple reason that it will not be possible on the field to ensure adequate control so essential for the welding technique.

Each Unit section of the structure manufactured in the Shop is complete in itself. All that would be necessary, after Shop manufacture would be to complete the structure by assembling together the various unit sections, and erection by rivetting at site. The economy and speed that would result from this method is obvious.

The 'Flow-Line' System :—Further economy and speed is achieved by adopting the 'Flow-Line' System for the manufacture. The 'Flow-

Manufacture of Gates and Shutters at Tungabhadra.

(Art Supplement)

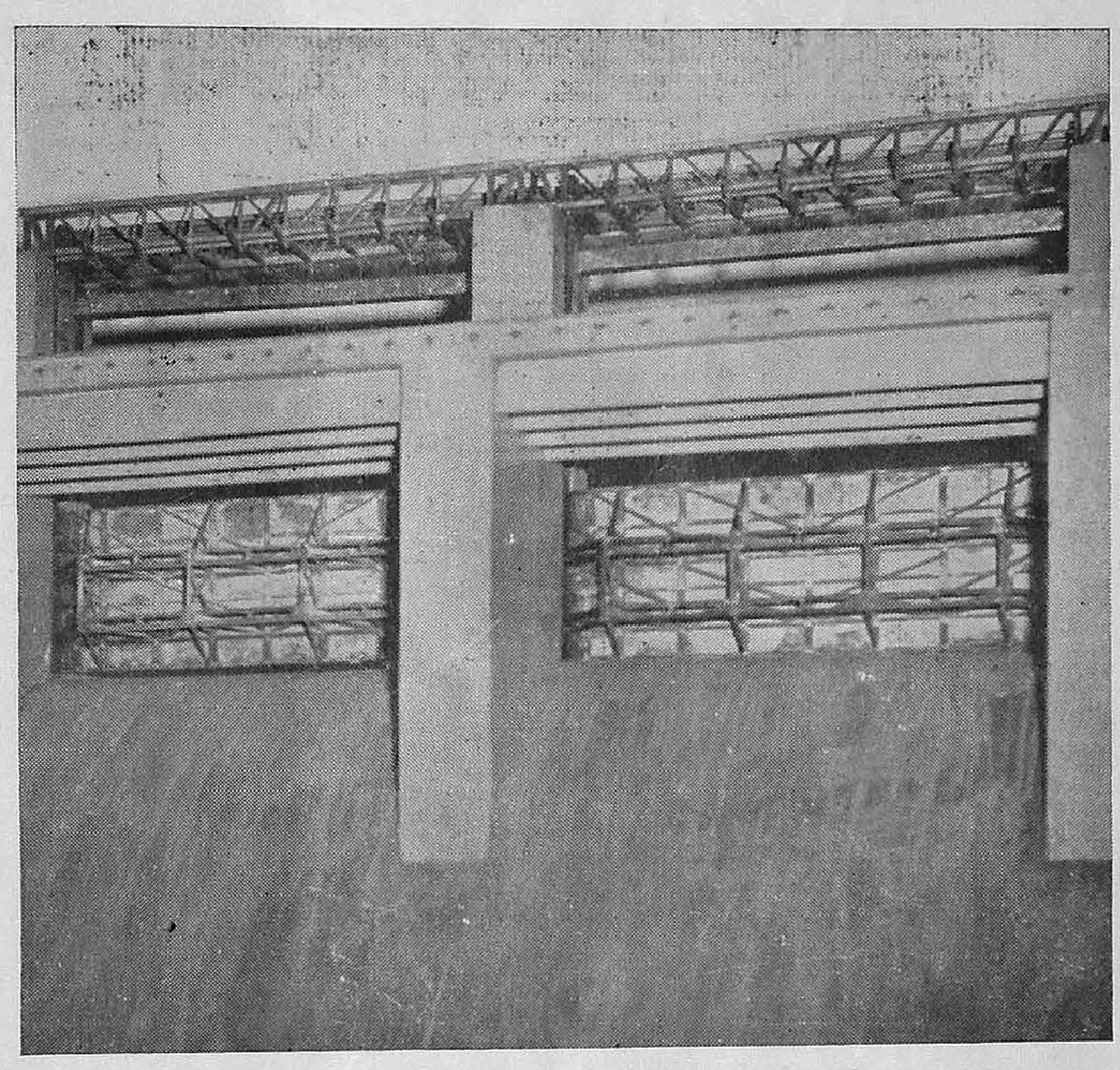


Fig. 1. How the Tungabhadra Dam will look when the gates and the operation bridges are in position.

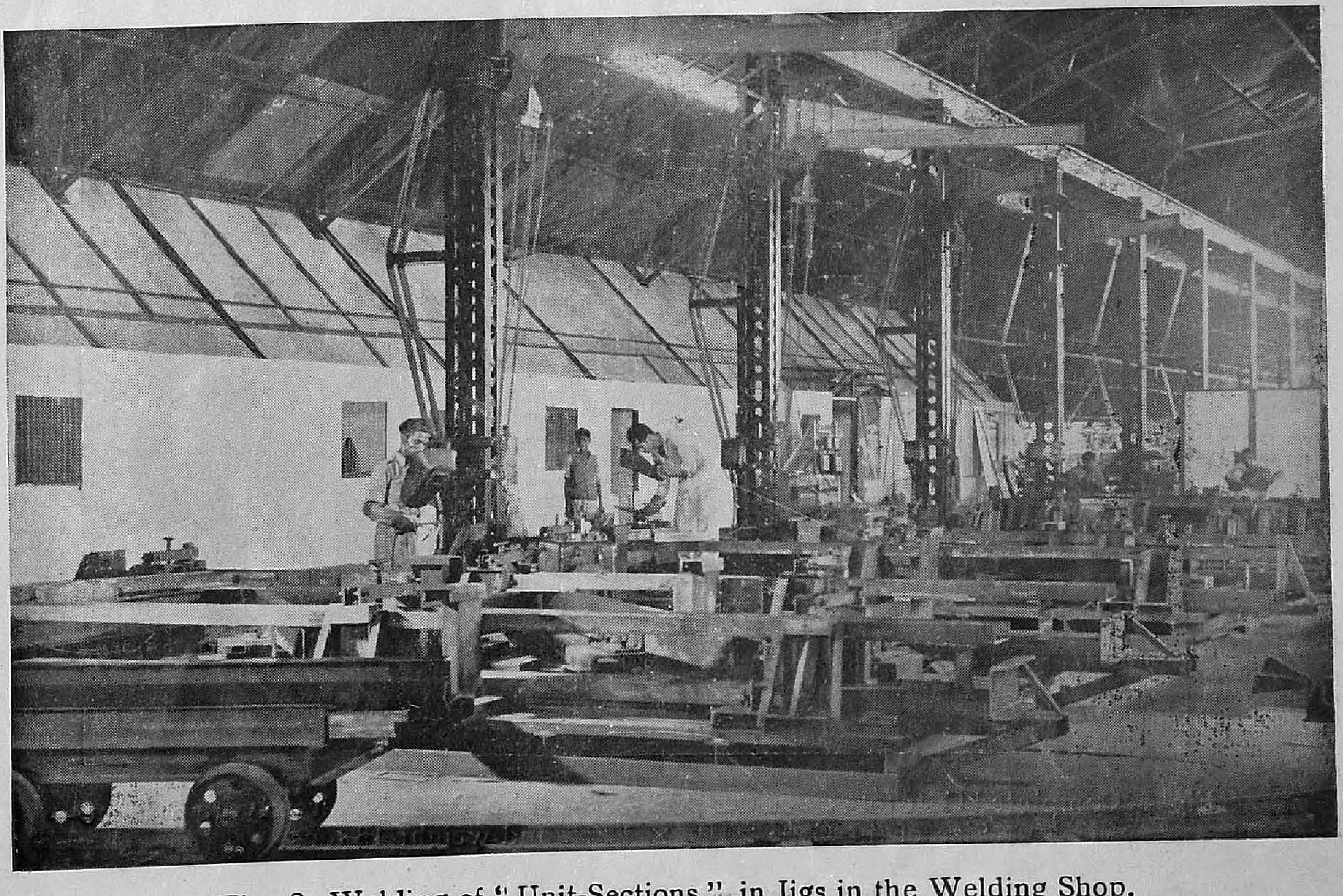


Fig. 2. Welding of " Unit-Sections " in Jigs in the Welding Shop.

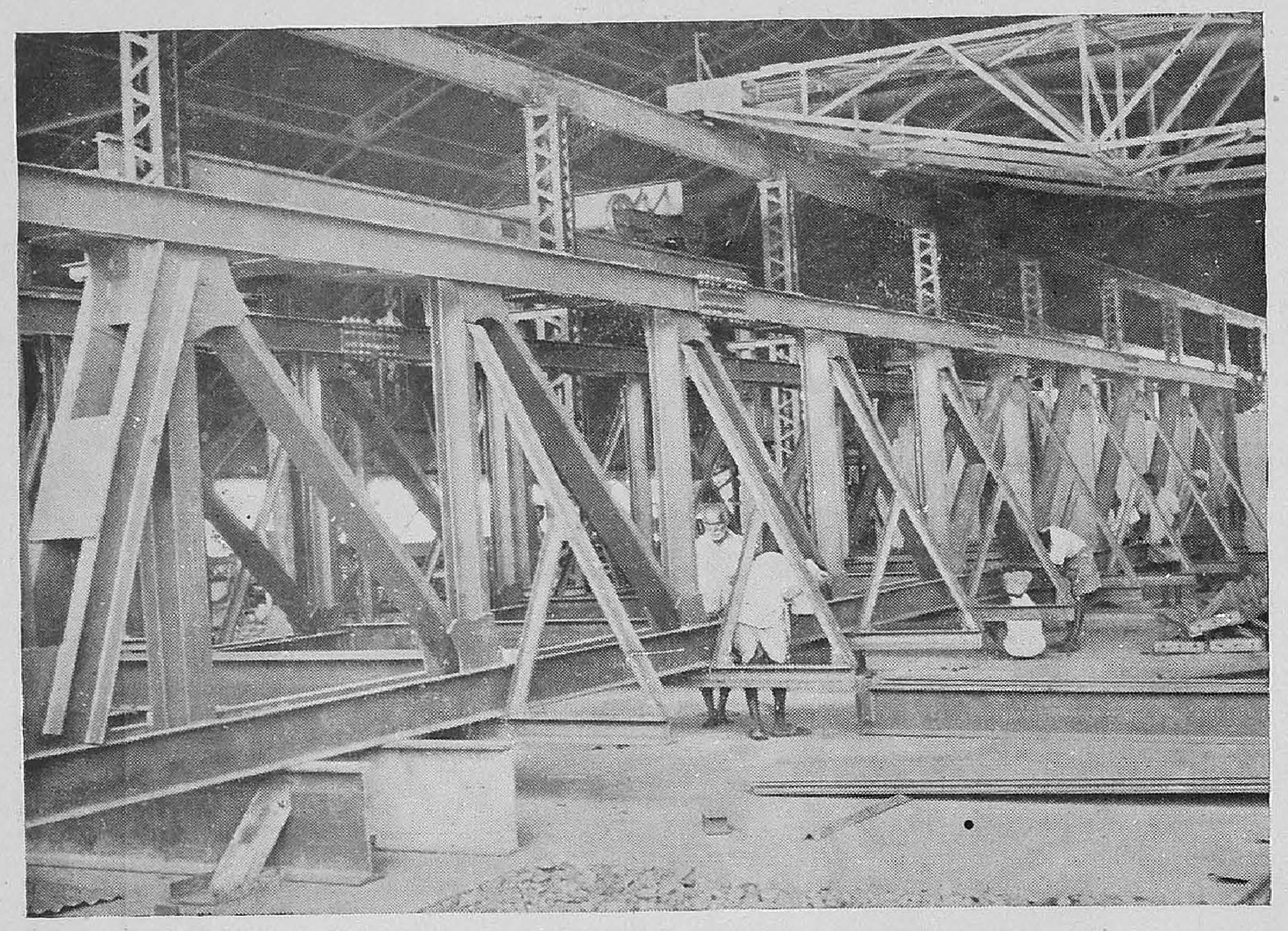


Fig. 3. A completed Bridge in the Assembly Shop.

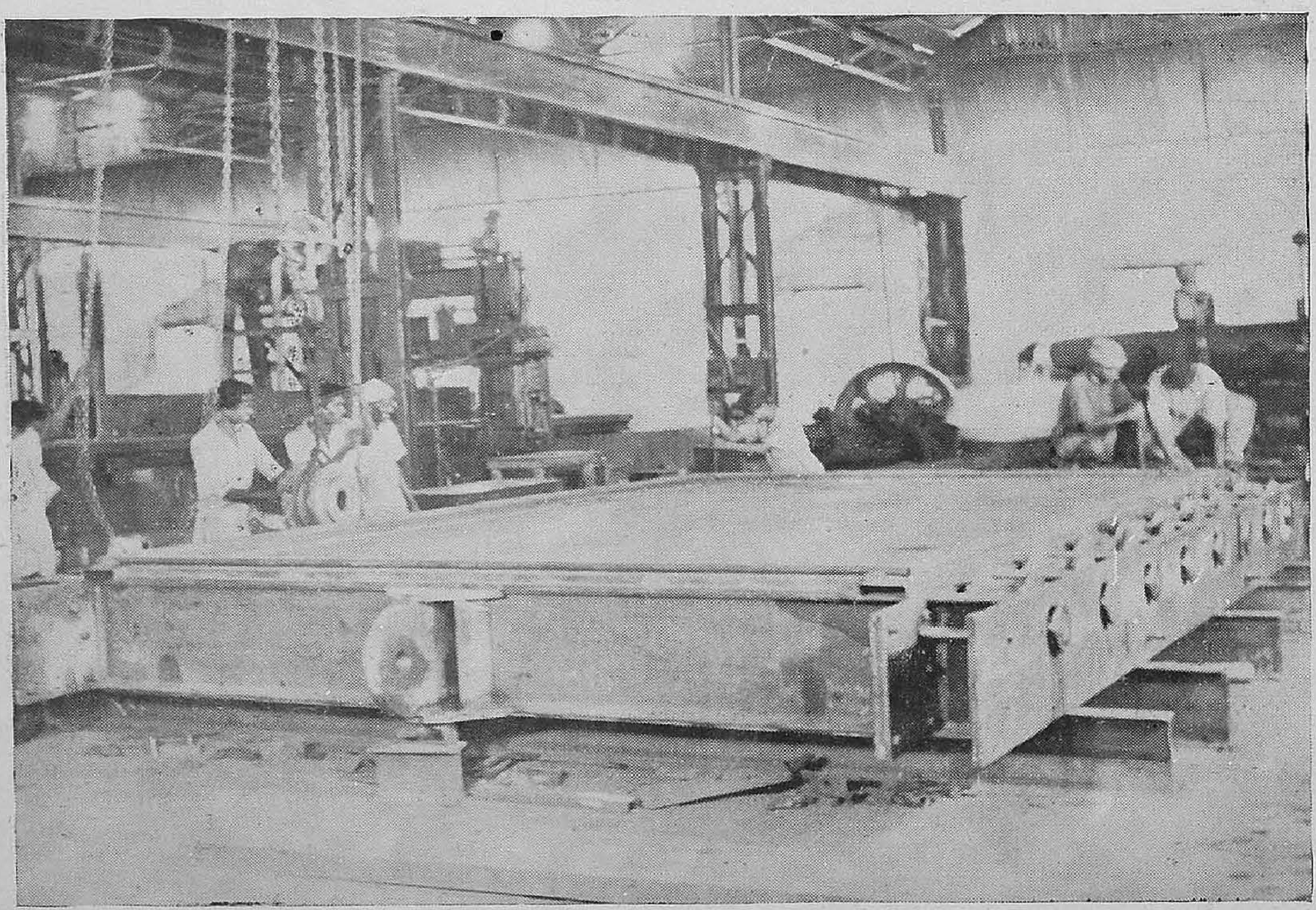


Fig. 4. Completed Gate for one of the penstocks of the Tungabhadra Hydro Scheme—
Size of the Gate 20 ft. × 12 ft. and weight 18 tons.

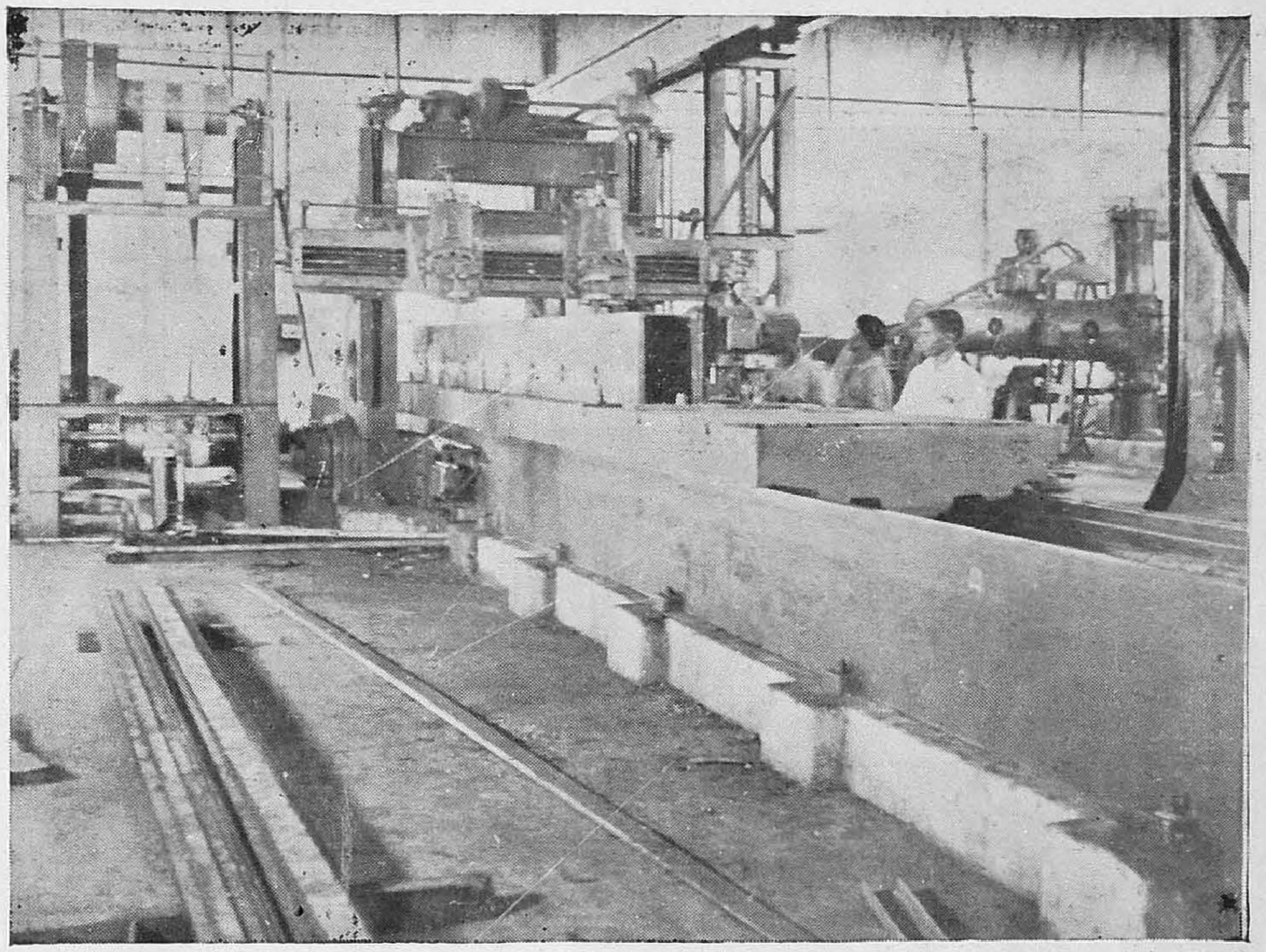


Fig. 5. End section of a Penstock Gate on a 64 foot long Planing Machine

Fig. 6. The picture shows the ball bearing, axle etc., of the wheels being examined by Sri M. S. Thirumalae Iyengar, B.E., M.I.E. (IND.), I.S.E., Chief Engineer for Tungabhadra Project.

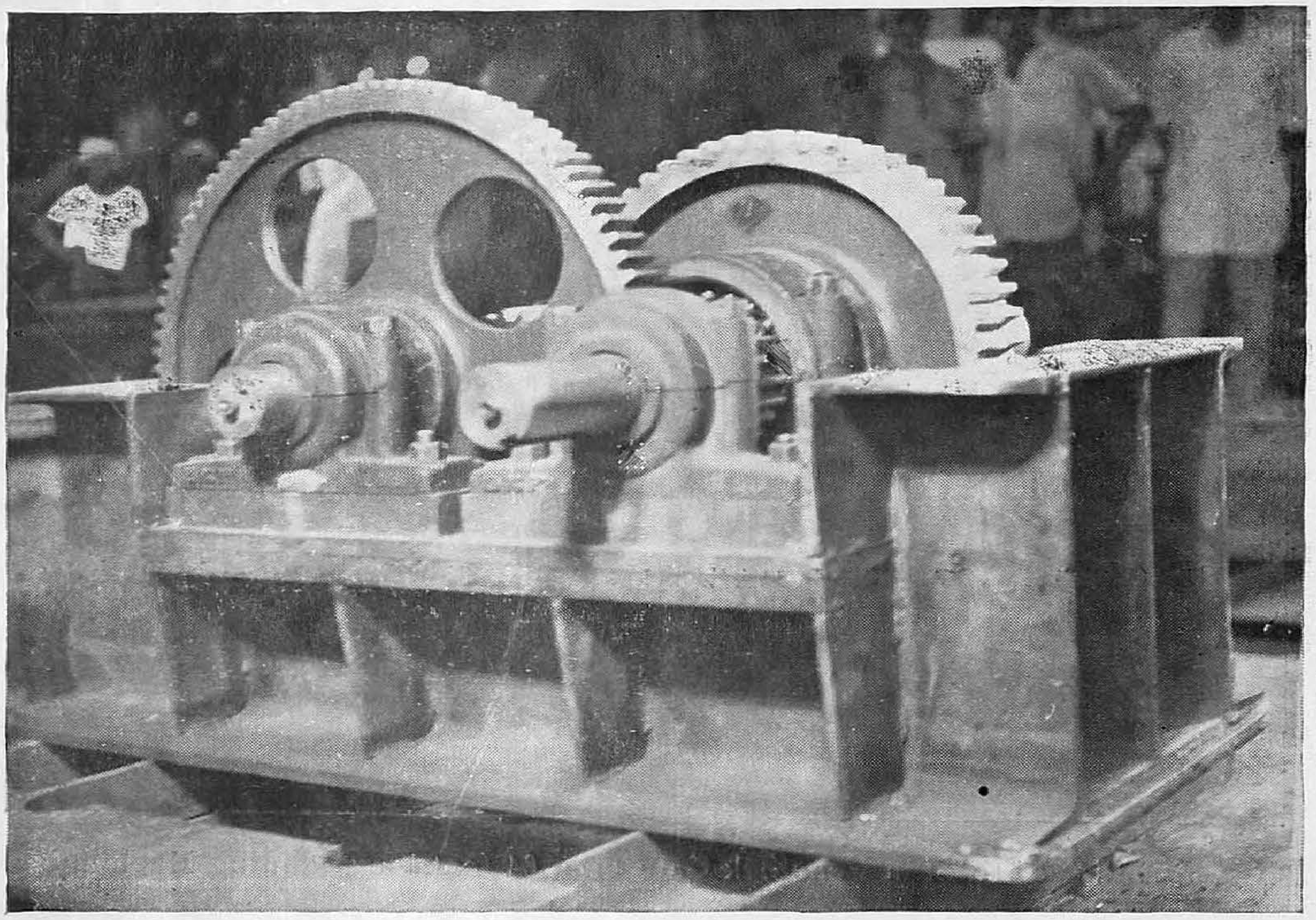


Fig. 7. Oil-bath gear box being assembled.

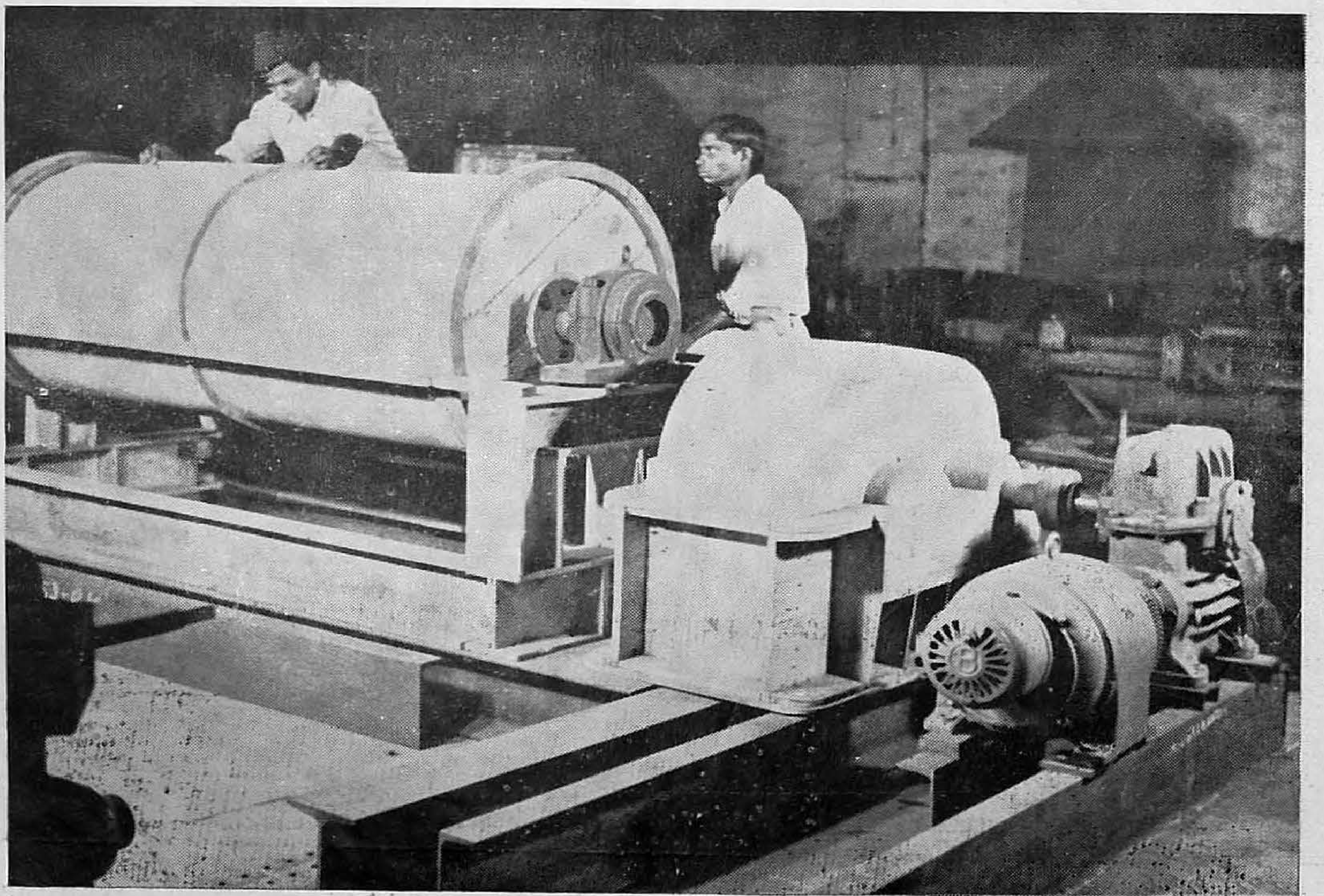


Fig. 8. Drum hoist of 35 ton capacity in the Assembly Shop.

Line' system of manufacture is the combination of the channelised flow of materials on the one hand, and of the controlled operational sequence on the other.

Right from the raw material yard upto the finished product the movement of materials from one stage to the other is in accordance with the planned requirements. In this system any discrepancy is easily detected and promptly rectified. Thus, smooth working is ensured at all times without interruption.

In the same manner, the set up of the equipment and the arrangement of the workers and supervisory personnel ensure efficient and systematic handling at all points, so as to minimise the likelihood of the finished product being faulty.

Sequence of Operations

(*A*). *Preparing the materials* (*Shop* '*A*'):—Standard Sections are cut to close limits of accuracy in Circular Saws or Band Saws, each of them being provided with firmly located stops. These are then passed through Hydraulic-Straightening-Machines of 50 Tons capacity for making them straight. Gusset Plates are cut to special "Stream-lined" shapes in the Profile Cutting Machines. Eight to ten Gussets are cut at one time by pressing the plates into a stack in a 100 Ton Press.

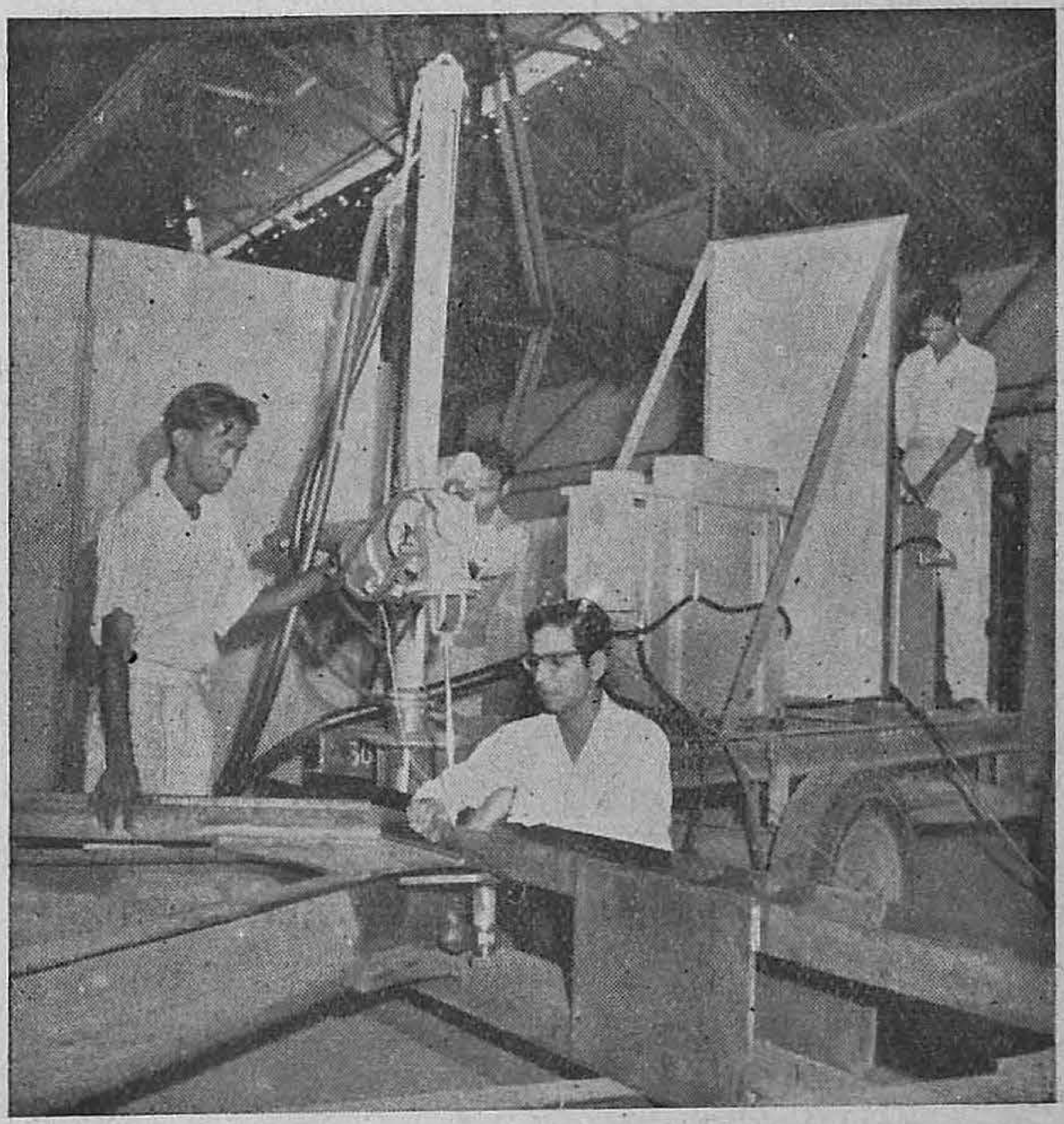
The Sawing Machines are fed by means of Overhead Cranes. The cut-pieces are handled by revolving floorcranes for feeding into the straightening machines. There are three such floor-cranes and each of them can operate in a full circle of 30 feet diameter.

(*B*). *The Welding Shop* (*Shop* '*B*'):—The materials processed to the required sizes in Shop 'A' are checked and passed on to Shop 'B', i.e. the Jig and Welding Shop, where the different pieces are welded into 'Unit-Sections' not exceeding 20 feet×10 feet in size or five tons in weight.

This Shop is designed for loading the Jigs twice a day. The materials loaded in the trucks in Shop 'A' are ready at hand in Shop 'B' for loading the Jigs. Thereby these loaded trucks serve the purpose of "Buffer-Stock". To facilitate quick loading of the Jigs from out of the buffer-trucks, Shop 'B' contains a net-work of tracks similar to a Railway Marshalling Yard.

Each of the Jigs is provided with its own pillar derrick which can handle pieces upto three tons in weight and swing through an arc of 150 degrees. Each piece when placed into the Jig gets located in its correct position by means of guides, which are firmly fixed to the Jig. Likewise, the gusset plates are positioned correctly by means of boundary-stops to suit the shape of the "Stream-Lined" gussets.

By adopting these methods, in addition to the dimensional accuracy of each piece being ensured, there is also safeguarding against a wrong piece not getting itself loaded into the Jigs.



X-ray apparatus in position for taking photograph of a welded joint

The next operation is welding the different pieces together according to a pre-determined sequence. The welding is controlled in such a manner that the length of weld deposited for each electrode is measured and recorded for each Welder. The record is made on Welding Control Cards; and later the data is transformed into graphical form to indicate the performance of each Welder.

Quality control:—The primary object of Quality Control is to ensure the soundness of Welding. The procedure consists of four items:–

(i) Preliminary Tests of the design-welds in a testing machine.

(ii) Visual checking and measuring the welds for each joint.

(iii) Periodical testing of the Welders to make sure they have not deteriorated on account of fatigue or other causes.

(iv) Taking X-Ray photographs of selected welds. The X-Ray apparatus used is a Philips, 150,000 Volts Machine capable of penetrating through two inches of steel.

The finished product (Shops 'C' and 'D'):—Each "Unit-Section" coming out of Shop 'B' is passed on to the Shop 'C', the Assembly Shop. In the Assembly Shop the various 'Unit-Sections' are placed into pre-determined positions on a platform, where they are adjusted for alignment and camber. Holes are drilled at suitable points for connecting the "Unit-Sections" together by means of splicing plates.

Next, the structure is passed on to Shop 'D' for painting. In this Shop rust and scales are removed by means of rotary brushes operated by flexible-shaft equipment. The units are then given one coat of red-lead paint; and suitable erection marks are then placed on each 'Unit-Section'.

Final tests:—The structure as a whole is then given a full-load test using Hydraulic Jacks for applying the load. Measurements are made by Strain Gauges and other devices for finding the general behaviour of the structure under load.

Quality control at various stages coupled with the final load tests will, it is believed, reasonably ensure the soundness of the structures manufactured in this Factory.

Cost of production:—Much of the equipment of the Factory was manufactured at the Factory itself; some of them are:—

(1) the 67-foot Span overhead Crane,
(2) Revolving Pillar Cranes operating in a 30-foot diameter Circle,
(3) Hydraulic Straightening Presses of 50 tons capacity and
(4) Three ton Pillar Derricks.

This has resulted not only in considerable economy in the Capital outlay on the Factory, but has also made it possible to complete the Factory in 12 months time.

The cost of manufacture including all the materials may range from Rs. 800 to Rs. 900 per ton depending on the weight of the structure.

This cost includes interest and depreciation at 5 per cent and 10 per cent respectively on the capital outlay of Rupees eight lakhs.

Manufacture of High Pressure Sluice Gates.

Another important activity of the Shutter Manufacturing Factory is the Manufacture of High Pressure Gates. From the Reservoir, the water is let into the Canals through Sluice Openings, that are built into the lowest positions of the Dam. Each sluice opening is controlled by a Gate. These Sluice Openings are small when compared with the opening at the top of the Dam. However, as they are situated at considerable depths below the surface, the hydrostatic pressure that they have to withstand is considerable, and this may amount to as much as 200 to 400 tons depending on the size of the gate, for 100 ft. depth of water.

Sluice Gates for the Irrigation Projects in India were in the past imported from abroad; and the general impression was, that they could not be successfully made here. The Shutter Manufacturing Factory at Tungabhadra Project has now got out these Gates for the Tungabhadra Dam, and it may be said that the bottom of the mystery has been knocked out true and square.

The performance of these indigenous Sluice Gates will have, no doubt, to be watched with interest. However, there are indications that these Gates can be expected to give adequate satisfaction during the operations, for quite a long period. The economy effected in cost of the locally manufactured Gates is about 25 per cent.

Manufacture of Hoisting Equipment:—The Shutter Manufacturing Factory has also tackled the problem of manufacturing Hoisting equipments for operating Sluice Gates. Already there are in production three such Hoists of 35 tons lifting capacity of the electrically operated type. Special techniques using the welding process were developed for manufacturing the various devices that go to make the Hoisting equipment. These were the making of Steel Gear Wheels, Oil-bath Gear Boxes, Steel drums for the hoisting ropes, Electrically operated brakes, Dial Indicators, and controls for fine adjustment of the Gate position.

Here again, the saving in cost is over 25 per cent as compared to imported equipment of similar specifications.

Shutters for the Canals:—The Shutter Manufacturing Factory has also made a large number of Shutters and Operating Gear for the Power Canal and the Low Level Canal systems of the Tungabhadra Project. The larger sizes of these Shutters for eight and ten feet spans are of conventional design; they are however, made out of Steel of All-Welded Construction, instead of in wood.

This technique has resulted in striking economies in the cost of the Shutters as it has enabled standardisation and mass production of the various parts in special Jigs and Fixtures.

Other items required for the Construction and operation of the Tungabhadra Dam as well as its Irrigation Systems have also been made in this Factory. In every case, readily available materials went into the manufacture of the items and local talent only was employed and developed for tackling the most difficult problems.

Now that a large number of items required for the Tungabhadra Project have already been made at this Factory and considerable experience has been gained in the various techniques, it will be appropriate for this Factory to line up for production of similar equipments for the other Irrigation Projects in this Country.

The fulfilment of the vast Irrigation Projects under the five-year programme may well require the manufature of several items of the types discussed. It is hoped that the Shutter Manufacturing factory at Tungabhadra with its background and experience will be the forerunner for establishing similar factories in other parts of India.

HOW WOULD YOU DO IT?

Solution to Problem on Page 4.

Exposed to the southwestern sun, the pipe was heated many degrees each day and expanded down the mountain side along the line of least resistance. Then, when the pipe cooled in the evening, it contracted in the same direction, that is, downhill.

[" That certainly is not the entire solution and without a statement of the conditions of anchorage is no solution at all. Can you think of some ? Try "—Editor.]

Pile Driving Tests in Sandy River Beds

1. Introduction

1.1. Steel sheet piles of different patterns are widely used for the construction of Coffer dams and cut-off barriers in river beds where the depth of sand is considerable. As the sandy overburden varies in depth and size, it becomes necessary to study the relative performances of the various types of interlocking sheet piles manufactured by the several firms to enable the choice of the most suitable one being made. These tests are also useful in indicating special difficulties, if any, that may be met with in driving piles in particular river beds, their usefulness as seepage cut-offs and their economics generally.

1.2. This article deals, on broad outlines, with the pile driving tests carried out for the proposed Ramapadasagara Dam across the River Godavari at the Polavaram site and also for the Dam across River Pennar at Someswaram site. The observed penetration of the different piles and their relative merits are discussed.

1.3.1. These tests have been fully described in the Technical Reports submitted by the Officers who had carried them out. This paper gives brief extracts from these reports. After indicating the equipments used and the test procedure adopted, the ariticle gives a brief resume of the tests proper and of the conclusions drawn therefrom.

1.3.2. Reference to the pile driving operations recently carried out at the Vijayawada Anicut is given at the end of the paper.

2. Equipments Used

2.1. The main items of equipment used in a typical pile driving test were the following :—

1. A Scotch derrick of 10 tons capacity.
2. Mckiernan Terry Hammer No. 5 9B3.
3. Mckiernan Terry Hammer Piles adopter.
4. Pile cap.
5. Air Compressors of 315 Cft. capacity.
6. Mckiernan Terry Extractor.
7. A Welding Plant.

2.2. Larsen, Bethlehem, Frodingham, Carnegie and Tata piles were used for the tests. Sketches of these piles are given in Fig. 1.

2.3. A brief description of some of the special equipments is given below :—

(i) *Scotch Derrick* :—This has a steel framed jib 69 ft. long, capable of moving in a radius of 45 ft. and lifting upto 10 tons–the working radius for this being 43 ft. The jib can rotate round a pivot pin securely fixed to a firmly built up foundation. A vertical post and two slanting posts with provision for loading the base girders on the

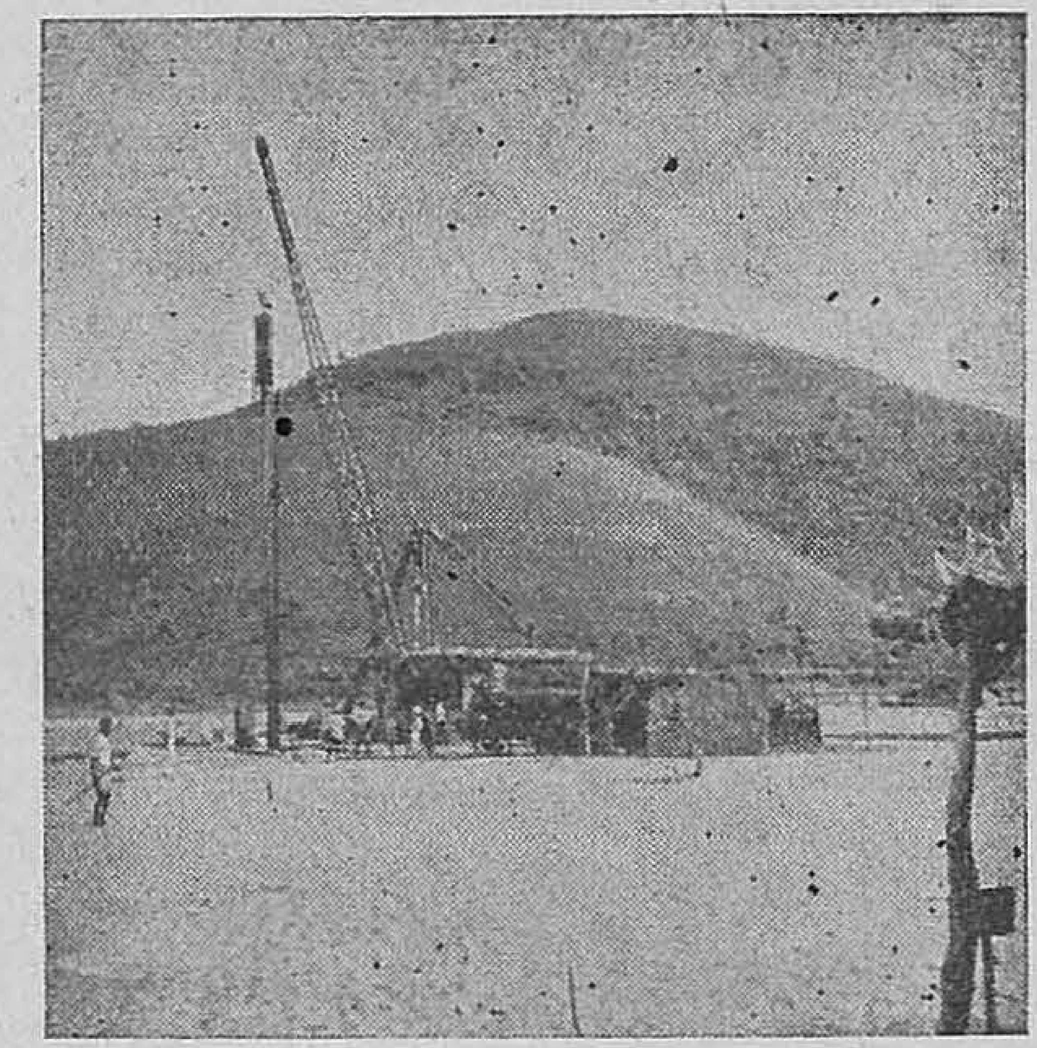
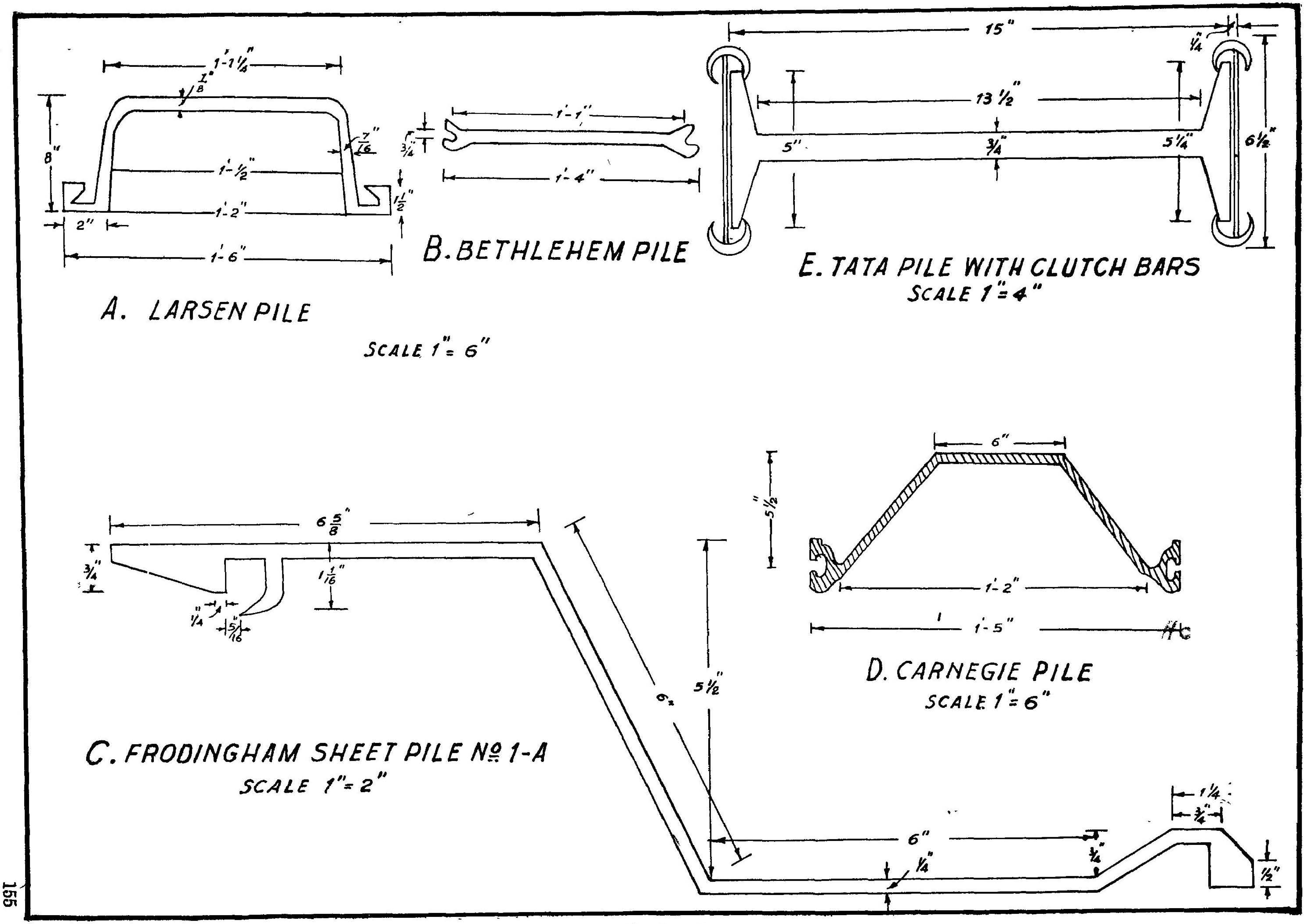


Fig. 2

offside to balance the jib, wire ropes and pullies, drums, gearwheels and handles are also provided. The derrick is used for hoisting the piles, lifting the hammer or extractor, bringing it to exact position and holding it during operation. Fig. 2 gives a general idea of the derrick and its assembly.

(ii) *Pile Hammer* :—The McKiernan Terry double acting pile hammers are sufficient for driving sheet piles in sandy river beds. These are operated on compressed air or steam. Its main advantage is that the rapid succession of powerful blows given by it keeps the pile constantly in motion. It can be suspended from the jib of a crane or derrick and no special pile driving frame is necessary. The 9B3 Hammer has a capacity of 600 cft. and gives 140 strokes per minute at 100 lbs/in ^{2} pressure.

(iii) *Drive cap or Special Anvil Block* is an attachment to the pile driving hammer at the striking end. This will be bolted on to the four legs of the pile hammer so that while striking, the full force of the blow is administered to the


A. LARSEN PILE
SCALE 1" = 6"
B. BETHLEHEM PILE
E. TATA PILE WITH CLUTCH BARS
SCALE 1" = 4"
C. FRODINGHAM SHEET PILE No. 1-A
SCALE 1" = 2"
D. CARNEGIE PILE
SCALE 1" = 6"
1'-1 1/4"
8"
1'-1/2"
1'-2"
2"
1'-6"
1'-1"
1'-4"
15"
13 1/2"
5"
3/4"
5 1/4"
6 1/2"
6"
1'-2"
1'-5"
5 1/2"
6 5/8"
1 1/16"
1/4"
5/16"
3/4"
1/2"
1 1/4"
Fig. 1

pile below. The cap is merely a thick steel sheet with side holes for bolts and nuts for attachment to the hammer. The use of the anvil block eliminates the possibility of the hammer sliding on the pile or the legs sitting on them.

(iv) *Pile Adopter* :—This is an attachment to the pile. It is a thick sheet of steel with solid blocks of iron at the bottom to suit the shape of the pile. For each type of pile a specially made adopter must be provided. For, generally, piles are of thin section and when blows are administered on them direct by pile hammers, the top of the piles may get themselves bent and deformed. This causes difficulty when a subsequent pile has to be attached to it for lengthening purposes. To avoid this, and for effective distribution of the force of blows on the webs and flanges, the adopter is used.

3. Tests at Polavaram Dam Site—First Series

3.1. In 1948, extensive pile driving tests were conducted at Polavaram in the Godavari bed for the Ramapadasagar Project. Two series of tests were made. The first series of experiments was conducted by using the Mckiernan-Terry No. 9-B3 Hammer with a driving pressure of 100 lbs. Three types of piles namely, (1) Tata's universal joist piles 15" x 5" with clutch bars, (2) Larsen No. 2 piles (3) Bethlehem SP 7 piles were tested.

3.2. A series of fourteen Tata's piles were interlocked and driven down to 80 ft. when a large boulder was struck against and a new site had to be chosen.

3.3. In this new location a pair of Larsen piles was first driven to 40 ft. depth when the top got damaged. The damaged portion was cut off and new lengths added; the piles were driven to a further depth of 24 ft. when the top again got damaged. Fig. 3 shows the pile with the deformed top.

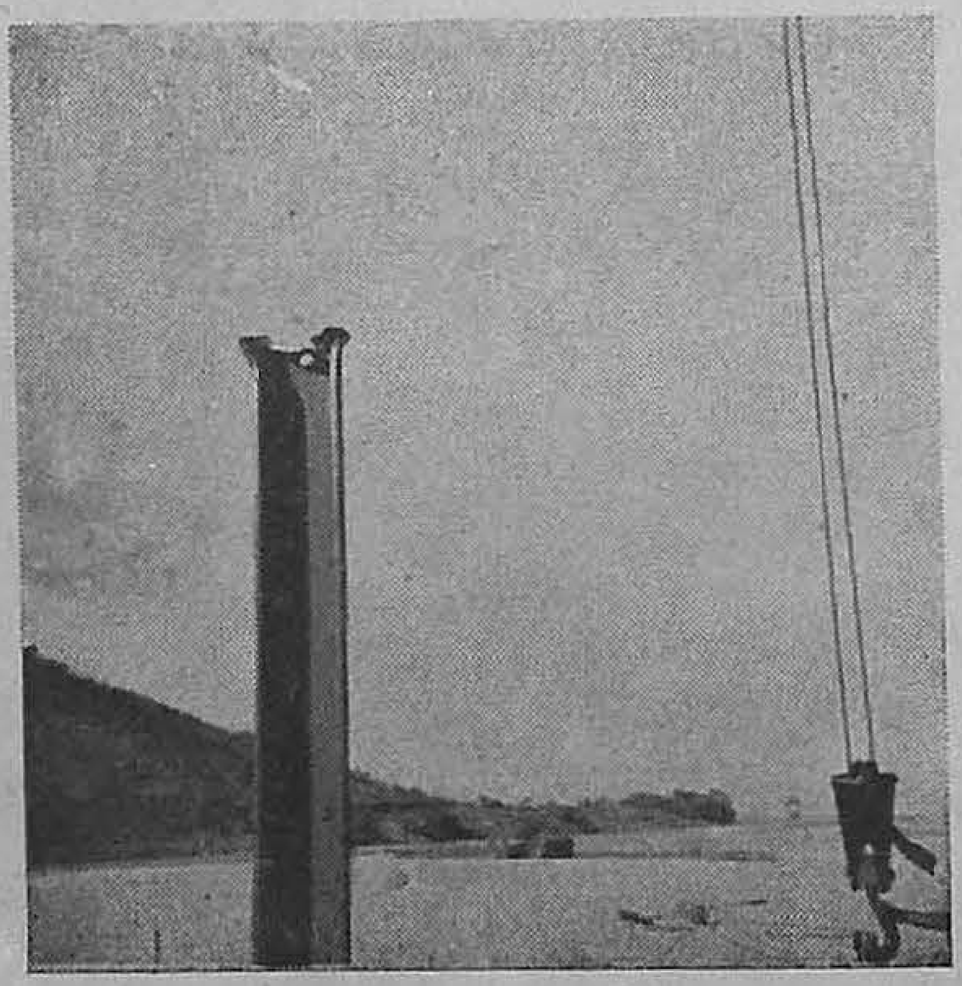


Fig. 3

3.4. The Tata's piles were driven down to 120' depth without the tops getting damaged. Next a pair of Bethlehem No. SP-7 piles were interlocked and driven down to 40 ft. without damage to the top.

3.5. The following conclusions were drawn :

1. The overburden at Ramapadasagar site could lend itself to driving in of sheet piles down to 120 ft. with a 9-B-3 Mckiernan-Terry Hammer.

2. The Tata's universal joist piles are more suitable where the piles had to be driven to great depths.

3. Larsen No. 2 piles cannot be used without protection to pile top by special caps or heads below 40 ft. depth.

4. Bethlehem piles could however be driven more easily down to 40 ft. and even more without their tops getting damaged.

5. Another interesting observation made in this test was that with Terry Extractor No. E4 it was not possible to pull out the piles driven to more than 70 ft.

4. Tests at Polavaram Dam Site—Second Series

4.1. In the second series conducted at Polavaram, steel sheet piles were driven, to form a circular shaft and pumping tests were carried out to observe the percolation effects, at the instance of Dr. Terzaghi, the Consulting Engineer of the Project. He directed that "as the available American sheet piles at site would be about 60 ft, a cylindrical shell should be constructed at a point where the rock surface is roughly 40 ft. below ground surface." He further instructed that attempts should be made to seal the underside of the piles with cement injections and remove the sand from the shell and see how far the cement seal was effective so as to frame detailed proposals for effective segregation.

4.2. A site where the bed rock was about 40 ft. below river bed was chosen. Two types of piles namely Bethlehem SP 7 (weighing 35 lbs/ft. run) 23 Nos. and Carnegie

piles M 116 (weighing 27 lbs/ft. run) 21 Nos. were assembled to form a cell of 18′–4½″ internal diameter. The piles were interlocked properly by using special hybrid piles at the junctions of the two types and supported by stiffeners and driven down to rock. A photograph of the pile driving in progress is given below (Figure 4).

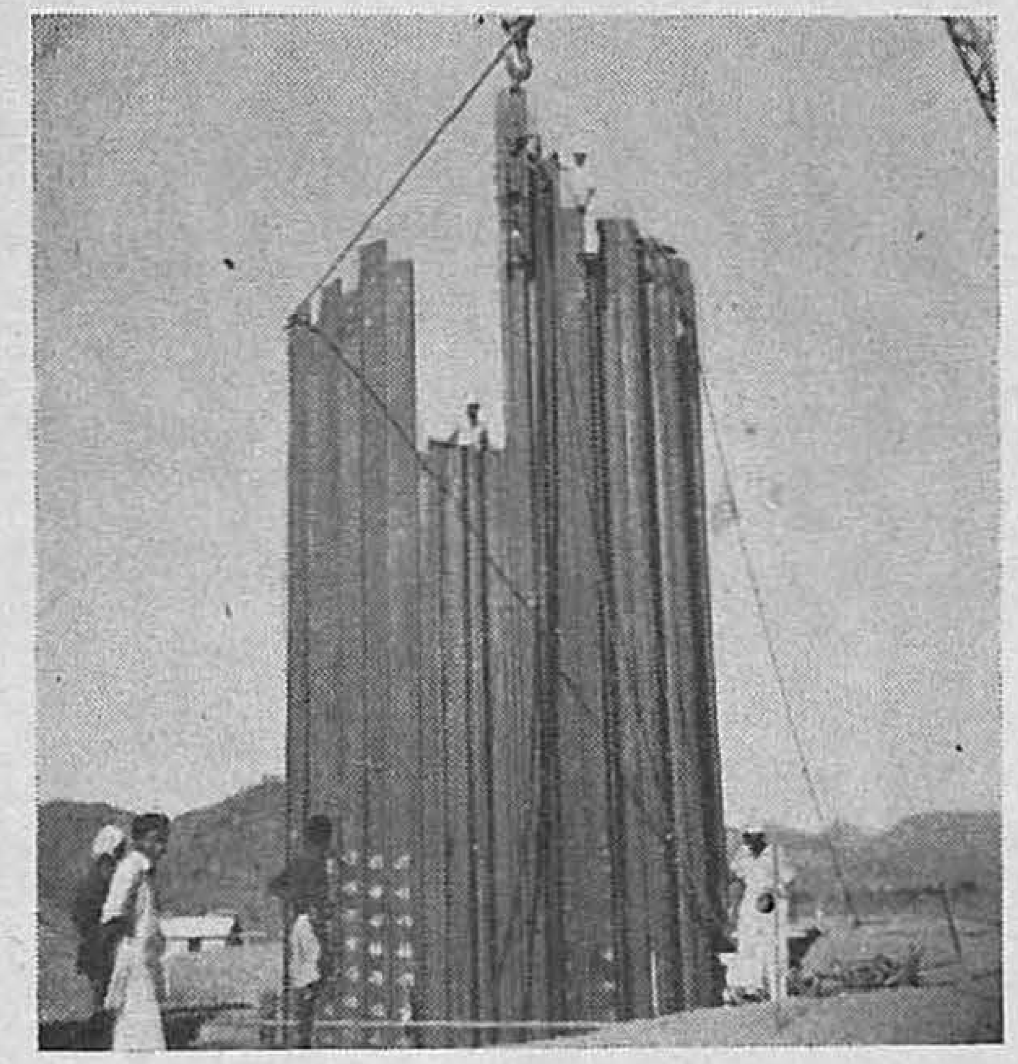


Fig. 4

4.3. The bottoms of the piles after driving in were cement grouted to seal the space between their bottoms and rock surface. The sand inside the cell was gradually removed with dredger buckets and water level depressed down by pumping. Compression rings and struttings were provided simultaneously. Dredging was done to depth of 27 ft. and the discharge at this depression was only 81 gallons per minute. The interlocks and grouted seal at the bottom functioned well in preventing any blowing in of sand under conditions of scour inside the cell.

4.4. From this series of experiments, it was inferred that—

1. Steel sheet piles formed into cylindrical cells can be easily driven down to 60 ft. depth without any damage to the interlocks in the overburden along the alignment of cofferdam.

2. The interspaces between the bed rock and the bottom of the sheet piles can successfully be sealed with cement injections so that in case the outside of the cofferdam is scoured, the sand from inside the cell will not run out.

3. 9B3 Mckiernan Terry Hammer is quite capable of driving these piles down to rock.

4. The steel sheet piles used on this job, namely SP7 Bethlehem and M. 116 Carnegie are tough enough to withstand the driving conditions anticipated here.

(A profile showing the relative positions of top and bottom of piles is given in Fig. 5).

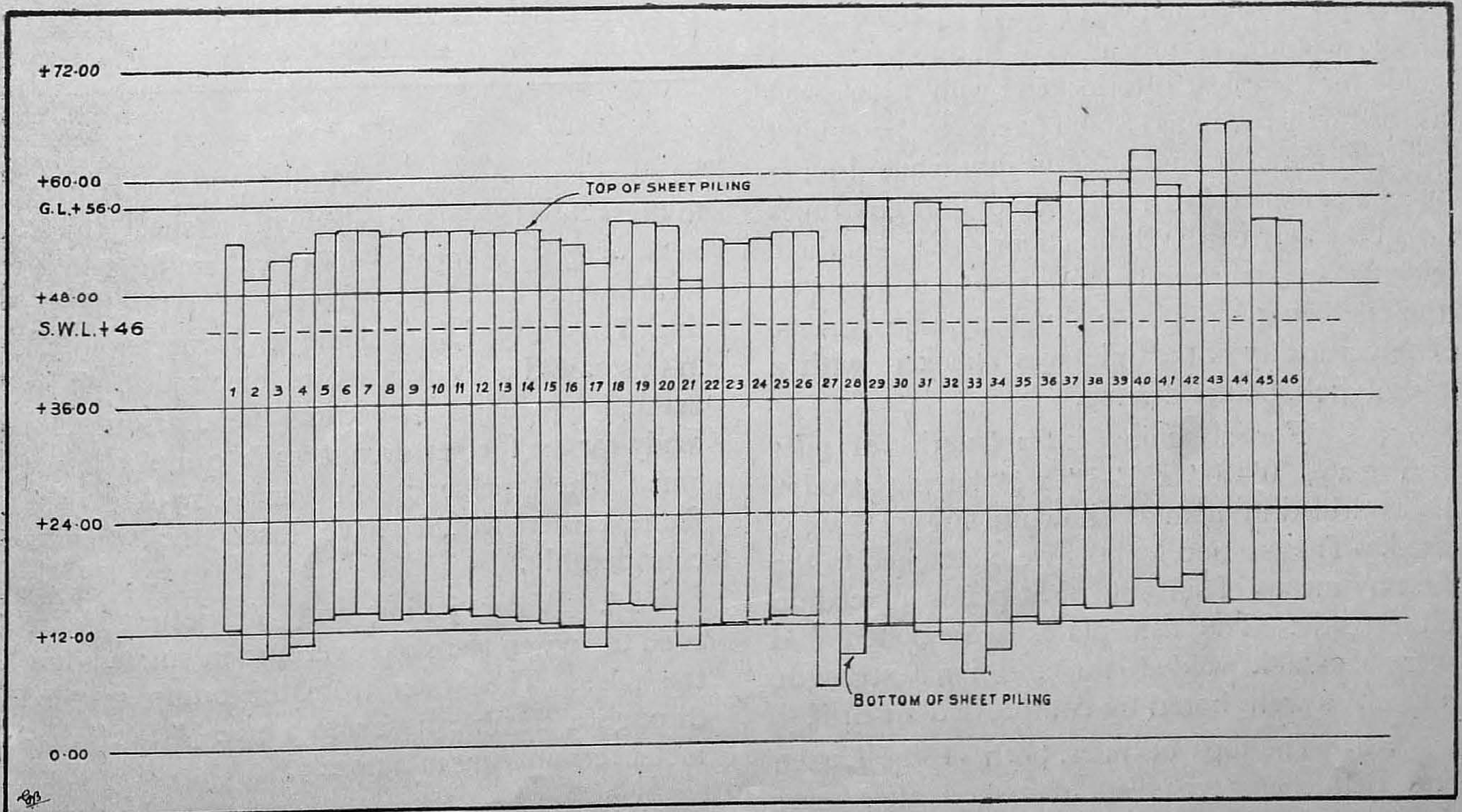


Fig. 5

5. Tests at Someswaram Dam Site—1950 Series

5.1. In 1950 and 1951 tests were made at Someswaram in the Pennar bed for the Kistna-Pennar-Project. As the test results from Polavaram were already available the test made here were of a only supplimentary nature.

5.2. *Tests made and equipment used*: The following tests were carried out:

1. One row of interlocked Larsen piles and one row of interlocked Bethlehem piles were driven down to bed rock about 70 ft. below bed.
2. One row of Larsen piles, one row of Frodingham piles and one row of Tata's Joist piles were driven down to rock about 70 ft. below bed.
3. A shaft, approximately 4'-0" square made of interlocking Frodingham piles and specially made corner piles, was driven down to clay about 40 ft. below bed.

Item (1) above was carried out in the summer of 1950 and items (2 and 3) in 1951. These are described in detail in the succeeding paragraphs.

5.3. *Pile Driving Operations*:—The piles were provided with holes through which wire ropes or hooks could be passed through. Then the pitching of the first pile was taken up. Every precaution had to be taken to ensure that it was absolutely vertical as it had to serve as a guide for the piles interlocked with it as well as for further lengths which were to be added to it during driving. Verticality while driving was maintained with the help of four guy ropes in all the four cardinal directions. Each of these guys was taken round a pulley so as to facilitate the pile being brought to plumb by adjustments. The verticality of the pile was checked with a plumb bob, before driving.

5.4. For attaching extra lengths of piles fish plates 0'-10" x 1'-1" x $\frac{3}{8}$" having twelve holes drilled in them as shown in Figure 6 were used. The second lengths were attached to the first by means of suitable fish plates placed on either side. The fish plates were bolted and were also tack welded for additional strength. The holes were bored by compressed air drill.

5.5. The tops of piles, both of the Larsen and Bethlehem types got damaged after being driven 30 ft. where pebbles and stones had to

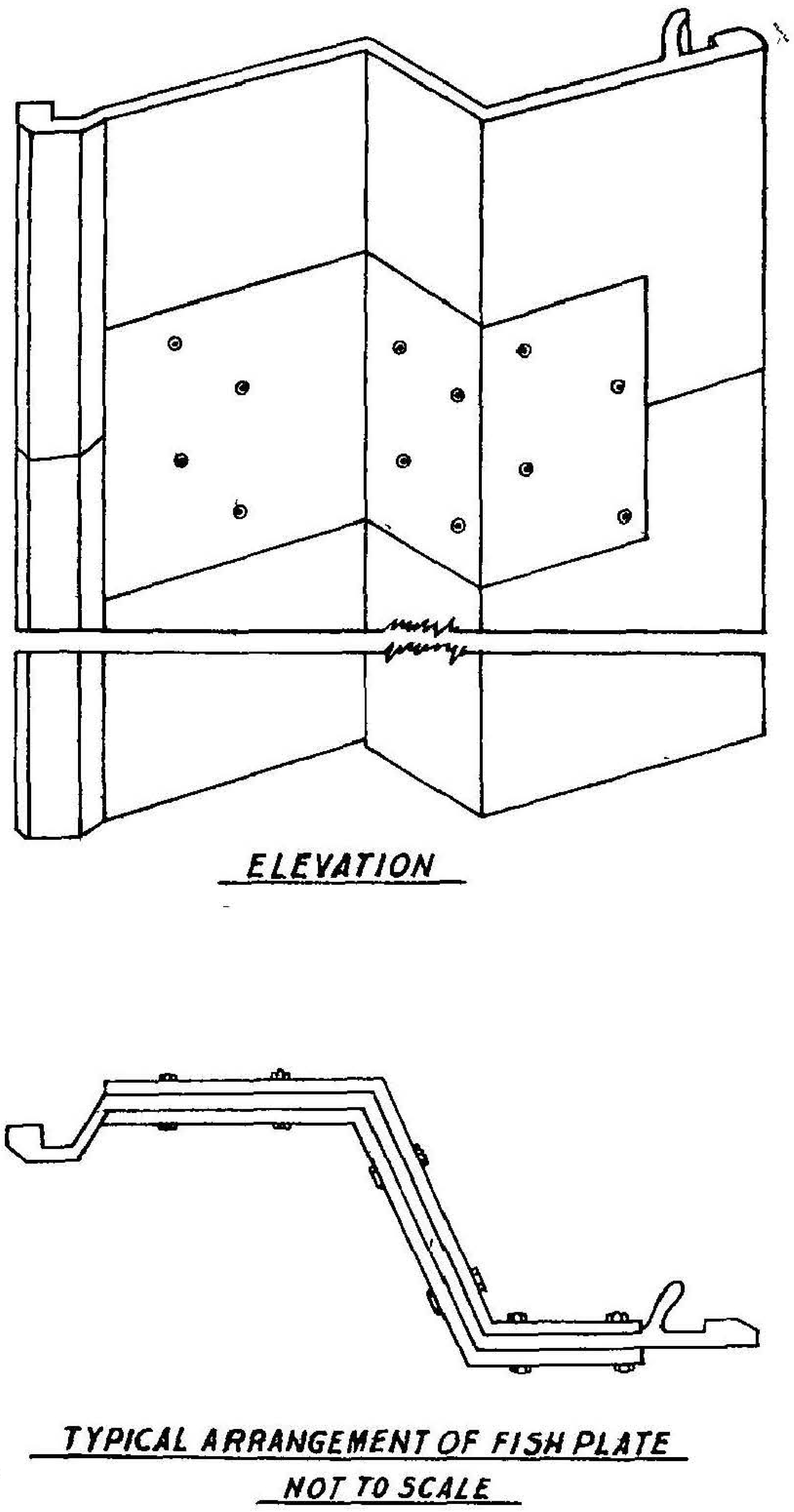


Fig. 6

be encountered against. Whilst it was very easy to dress the Bethlehem piles with a sheet thickness of $\frac{3}{8}$" by means of a welding machine, hacksawing had to be resorted to for the $\frac{7}{8}$" thick Larsen type, as welding was uneconomical in this case due to wastage of electrodes. This manual operation in setting the Larsen piles ready for the further driving took quite a lot of time. The presence of an Oxy-Acetylene flame cutter would have eased the situation considerably.

5.6. Water jetting and air jetting were tried to loosen the sand overburden surrounding the piles. The water from the pump was led through a flexible hose into a pipe $\frac{1}{2}$" to 2" diameter terminating in a nozzle.

5.7. There is no need to drive this pipe into the ground or to fix it to the pile when driving.

steel sheeting. The pump should be of the Duplex type delivering about 6,000 to 8,000 gallons per hour at a pressure of 120 to 150 lbs. per sq. inch. If the piling is to be a permanent part of the structure, depending on penetration for its stability, jetting should always be stopped a few feet before the piles reach their final level in order to ensure that the bottom of the piling is in undisturbed terrain.

5.8. Air jetting was also tried ; but it was not very successful because the pressure was not enough to aid the piles in breaking the boulders.

6. Salient features of the various piles used.

6.1. *Larsen piles*: Larsen piles (No. 5) were 18″ wide and weighed 65 lbs./ft. run. They were in lengths of 16 ft. When one pile was driven to 12′ depth, the next pile was interlocked with the first. This showed signs of breaking at top when driven to 30 ft. depth. It was easier to drive the Larsen piles in pairs. The Larsen piles could be driven only to 68½′ depth at the sets, probably due to rock being met with at that depth which could not be cut through with the equipment available. A Larsen pile, in situ, with interlocking arrangements is shown in Figure 7.

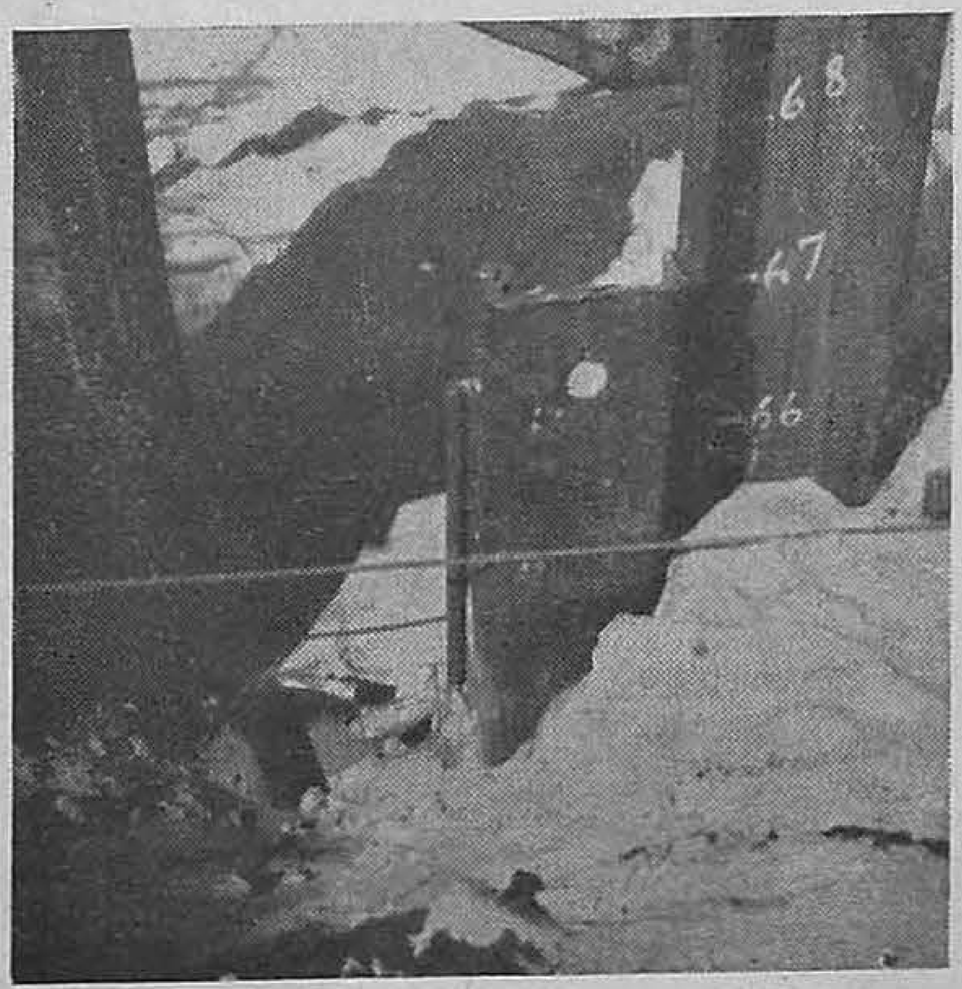


Fig. 7

6.2. *Bethlehem piles*: Bethlehem piles were 16″ wide and weighed 35 lbs./ft. run. As the pile was available in large lengths and as doubts were entertained as to its resistance to bending, four long channels of iron 5″ x 2½″ and 20′ long were attached to it, two on either side which acted as stiffeners. This pile went down faster than Larsen pile for the same number of blows. A Bethlehem pile showing the second length being added on to the first is shown in Figure 8.

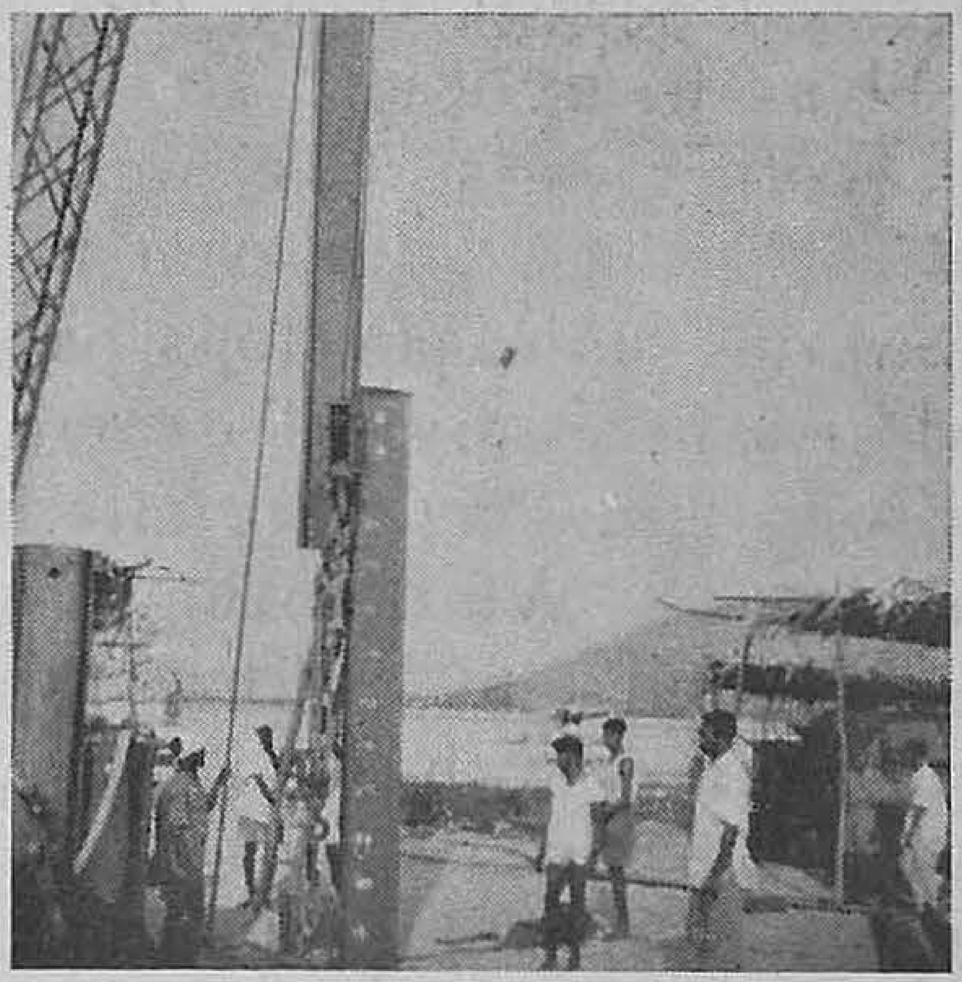


Fig. 8

6.3. *Frodingham piles*: For this, three piles were kept interlocked with each other in a straight line so that they formed a pile wall. The piles on the sides were driven to 19 ft. which served as guides when the central pile was driven.

6.4. *Tata's Joists*: Two clutch bars were attached on either side of pile proper. The joists showed a tendency to lean forward or 'creep' in the direction of driving. Hence driving was done intermittently each time, the swing to the sides being corrected. When one was driven to 27′ the other was made to slide through the clutch bar of the first. The clutch bars showed a tendency to go down along with the second pile when it was driven. So it was tack welded to the joist at a number of places but even then the weld gave way at many places. The second pile was driven down by attaching a new joist. In this way, in turns, the piles were driven down to 71 ft. when they were found to rebound.

7. Sheet Piles Box

7.1. Apart from these sheet piles, a four feet square box formed of Frodingham piles was driven in the 1951 experiments. This is described below :

7.2. *Frodingham pile with 4′–0″ square box*: The sheet piles as well as the corner piles each 16′ long were threaded on the ground and the box was formed. Then the whole box was lifted and placed in position on the ground and the box was corrected to the plumb. When the whole 16 ft. depth was driven, the next

length was then threaded and each pile was joined to the corresponding piles with suitable fish plates and bolts and nuts as in the case of stronger piles. But it was found that the corner piles got damaged badly due to absence of a suitable drive head. To obviate this defect the corner piles were welded to the adjacent straight piles. But even then beyond 32′ depth, fraying of the pile tops persisted and cutting became necessary. To overcome this, temporary bits 2 ft. long were attached to the piles during driving. When driving was over, the bits were removed and the tops were found undamaged. Even with all these precautions, the box could be driven only upto 42′ depth. Beyond this it was not possible to drive the box as the progress was very slow, taking in 1500 strokes per minute per foot of penetration and it was felt that even two compressors would not suffice.

8.1. The rough cost of driving several types of piles are given below:

Piles	Cost of driving straight piles per 100 square feet.	
	First 30 ft.	30 ft. to 60 ft.
	Rs. A. P.	Rs. A. P.
Bethlehem ...	975 0 0	1,125 0 0
Larsen ...	1,541 0 0	1,742 0 0
Frodingham ...	1,035 0 0	1,484 0 0*
Tata's joists with clutch bars. ...	800 0 0	1,440 0 0

8.2. *Driving piles of this type in a small box with complicated corner piles and close interlocks involved higher costs. For 100 sq. ft. of box the cost worked out Rs. 4,400 for the first 30 ft. depth and for the next 15 ft. it was Rs. 11,200.

9.1. *Conclusions*: The tests had been carried out under serious limitations due to want of adequate and proper equipment. Nevertheless the results are representative and valuable.

9.2. The tests at both the Polavaram and Someswaram sites indicate that there will not be appreciable difficulty touching the full depth of 160 burden *i.e.* 120 ft. in River Godavari and 150 ft. in River Pennar.

9.3. The relative merits of these piles are furnished below:

Type of pile	RANK OF		
	General Performance	Easy driving for first 30 ft.	Greater depths.
1. Tata's universal joists piles ...	1	1	2
2. Bethlehem ...	2	2	1
3. Larsen ...	3	4	Not Reliable*
4. Frodingham ...	—	3	

*These are designed for strength against lateral thrust and have maximum sections farthest from the Neutral Axis and are to be preferred only where this condition is required.

9.4. It will be seen from the above that where straight forward sheet piling is involved

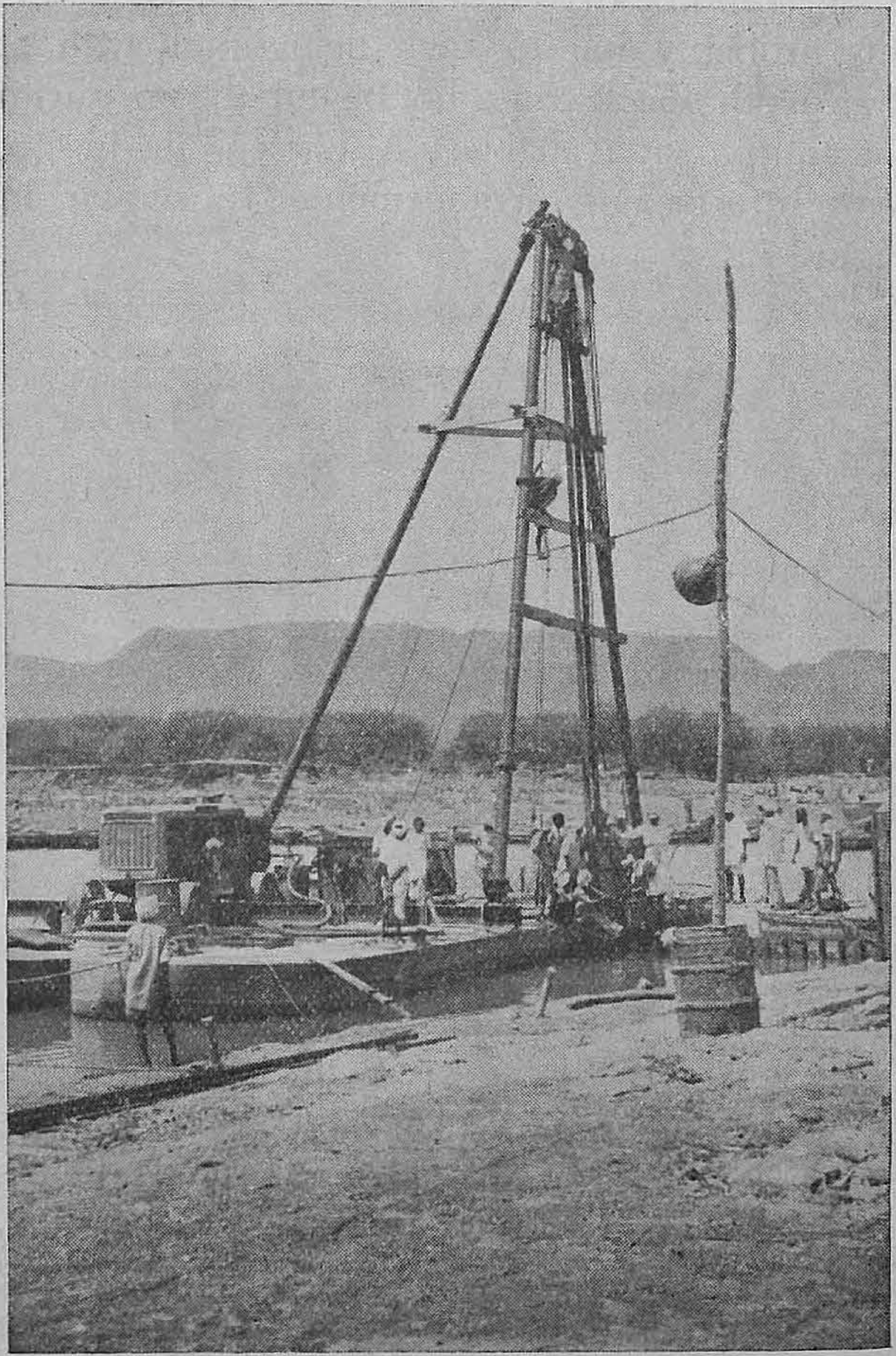


Fig. 9

any standard pile of the type of Tata's or Bethlehem's would be sufficient. If a pile of the type of Larsen No. 5 is chosen on account of cheaper cost and easy availability an allowance of 30 to 50 per cent over and above the routine piles should be added on to the cost of driving on account of the extra precaution to be taken.

9.5. For the box type, cellular form is better than the square, not only because of the saving in perimeter but also on account of the greater strength, and ease in driving piles. However, there is a limitation to curvature as too much of it will labour under the difficulties encountered in driving Larsen pile No. 5.

10. Pile Driving at Vijayawada Anicut

10.1. *Allied Works*: Before concluding, it may be stated that pile driving operations on a large scale has been completed in another place *viz.*, the recent Reconstruction of the Krishna Anicut. Fig. 9 shows the rigging up of the frame and the Mckiernan Terry Hammer No. 6. The piles were driven in front of the approach channel above the under sluices, the Sitanagaram lock and the Krishna Western Delta Main Canal. The construction methods and further details of this work are beyond the scope of this article and will be dealt with in another paper.

Compiled from the reports :—

1. Steel Sheet Pile Driving Tests at the Ramapadasagar Dam Site–1948, Report No. 3.
2. Sheet Pile Cell Driving Tests at the Ramapadasagar Dam Site—1948 Report No. 4.

   (By Sri P. Thambumalla Reddy, B.A., Assistant Engineer; Sri N. Hanumantha Rao, B.E., Executive Engineer; and Sri K. V. Ekambaram, B.E., I.S.E., Superintending Engineer.)
3. Kistna Pennar Project—Field Investigations for the Pennar Dam—Report No. 6.

   (By Sri P. Nagendra Rao, B.A., B.E., Junior Engineer; Sri N. K. V. Subba Rao, B.E., (Hons.), Assistant Engineer, and Sri T. P. Kuttiammu, B.E., A.M.I.E., Executive Engineer.)

IMPROVED WATER TREATMENT.

(*Continued from page 2*)

Electrochemical method is similar to chemical coagulation in that both depend on formation of insoluble aluminum hydroxide. But it forms the hydroxide directly from the metal with the aid of an electric current.

The treating equipment consists basically of an electrolytic battery, a sand filter, and an ionexchange unit. Battery and filter purify the water of its organic contamination, while the ionexchange unit conditions water for hardness or alkalinity, when required. A carbon filter (not shown) is also part of the system.

As current flows through the battery, aluminum hydroxide is generated. The hydroxide is added to tap water, which then flows to the sand filter. The hydroxide absorbs organic matter present in the water and is retained in the sand filter.

Particles of the hydroxide penetrate into the filter, cover the sand grains, and produce an activated filter. This impregnated filter is fast, as well as effective, because of its openness and an action largely physical and chemical, rather than mechanical.

Water-flow rate and the degree of treatment required determine the aluminum hydroxide needed. Varying the current flow regulates the generating rate of the hydroxide. Generally, current is less than 3 amp. and voltage about 5 volts. Hydroxide feed is usually less than 3 parts per million parts of water.

Operating cost is about 18 cents per 10,000 gal. of which the electrolytic cost is about one-fourth. Additional treatment for alkalinity or hardness, say for 3 gr. per gal. of $CaCO_3$ alkalinity or 7.5 gr. per gal. of $CaCO_3$ hardness, would cost about 50 cents per 10,000 gal.

—*With acknowledgements to " McGrew-Hall Digest ", January 1953.*

Progress of Research Work in Madras

[*Note* :—In previous issues of the Era *viz*., Volume III Nos. 1, 2 and 3, the activities of the Irrigation Research Station at Poondi were reported upon by Sri J. Visweswara Rao, B.E., M.Sc., A M.I. Struct. E. (Lond)., Assoc. M.ASCE, A.M.I.E (Ind)., Research Executive Engineer, Irrigation Research Station, Poondi.

Below is an article on the Soil Research Station by Sri T. N. Seshadri, M.A, MI R.E., Aff. Am. Soc. C. E., F. Inst. P., Physics and Soil Mechanics Officer.]

B. THE SOIL ENGINEERING RESEARCH STATION, CHEPAUK, MADRAS.

The Soil Engineering Research Station started to function in 1946 when the present Physics and Soil Mechanics Officer was appointed to organise and build up the station. Judging objectively from its record of work since its inception and the written impressions of top-ranking technical visitors to the Laboratory, the Research Station has done a creditable and impressive bit of useful work. " We cover a much larger floor area in the Bureau of Reclamation Laboratories but you cover the same area of research, and I feel you cover certain areas in even greater detail"—so complimented an American expert who recently visited the research station.

The work of the station, however, is carried on silently and without ostentation and in a spirit of real service. The Laboratory has shunned every opportunity for even a mild publicity, so much so that not many know of the existence of this Laboratory itself, and even the few who know of its existence do not fully realise the intensely practical nature of its work and the degree of help it affords to the department in the design and construction of earth dams and embankments.

The work of the Laboratory has been standardised to such a degree, and reduced to a routine so thoroughly, that the Laboratory appears to run almost by itself. As much as six soil samples during every single working day are tested in the Laboratory for all the significant properties like mechanical grading, plastic and shrinkage factors, water-tightness, cohesive strength, angle of internal friction, unconfined compressive strength and compaction characteristics. The test procedures have been drawn up and standardised in a comprehensive manner. The forms for recording the experimental data and the final test results have been designed with an eye towards ultimate utility : and the necessary graphs are incorporated in the forms themselves. Tabular forms and nomographs have been developed to minimise the drudgery of repeated calculations. Provision is made for checks and counter checks so that mistakes due to casual slips in calculation are eliminated. The whole organisation is pitched towards coordinated efficiency : and the work of the Laboratory hums so smoothly that a casual visitor finds it difficult to realise that within its walls more than 2,000 samples per year are tested thoroughly and completely for all their significant characteristics —a rate attempted by few laboratories of its kind anywhere.

Sri S. L. Malhotra, the then Secretary of the Central Board of Irrigation and Power, after visiting the Laboratory during February 1950 complimented it on its developing " Standard methods for sample tests " and requested a " Note for the Central Board of Irrigation and Power on the standardised work, for use in other Soils Laboratories " in the country. In response to that request a full account of the daily routine of work in the Soils Laboratory, along with the detailed procedures adopted for the individual tests was written up and included in the Station's Annual Report for 1950. A reference to that paper would give a complete and clear account of the test work carried out in the Soils Laboratory. A flow sheet of the daily routine of work in the Soils Laboratory is reproduced as Figure 1 and may be found interesting.

Of late quite a few outside agencies, like the Great Indian Peninsular Railway, the Khandla Port and Cochin Harbour authorities, the Madras Port Engineers, the Madras Electricity Depart-

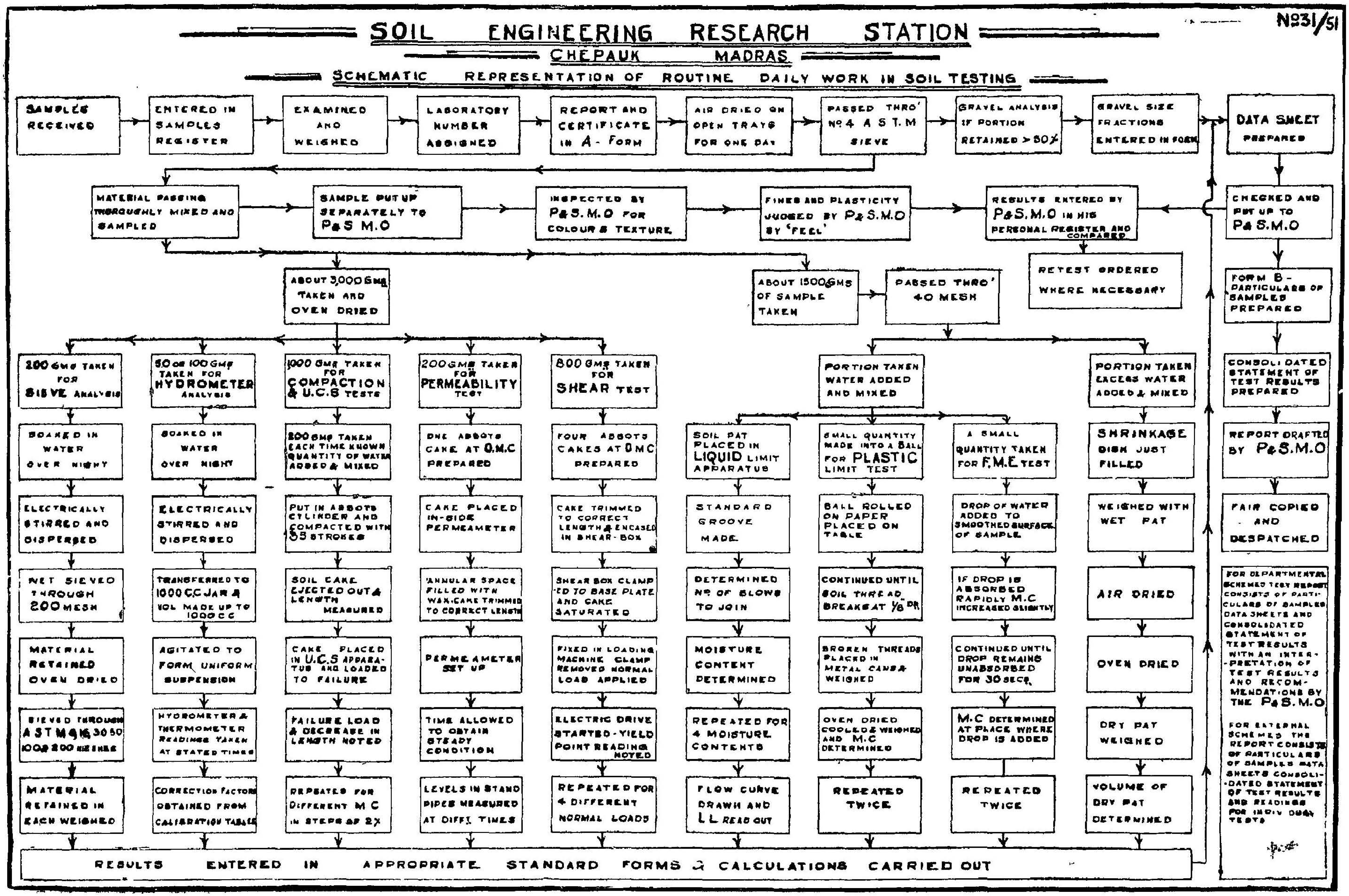
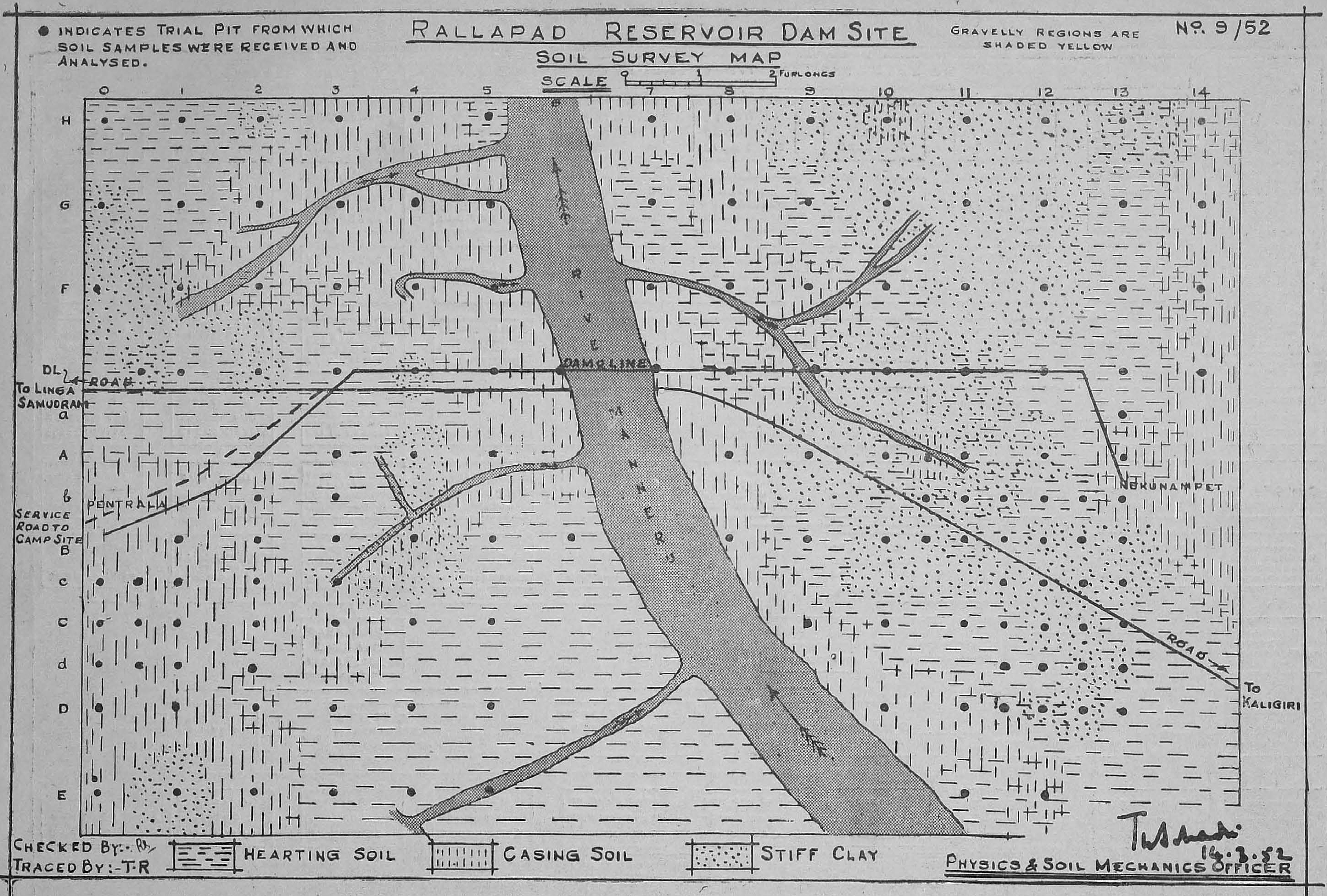

SOIL ENGINEERING RESEARCH STATION
CHEPAUK MADRAS
SCHEMATIC REPRESENTATION OF ROUTINE DAILY WORK IN SOIL TESTING
No31/51
SAMPLES RECEIVED
ENTERED IN SAMPLES REGISTER
EXAMINED AND WEIGHED
LABORATORY NUMBER ASSIGNED
REPORT AND CERTIFICATE IN A-FORM
AIR DRIED ON OPEN TRAYS FOR ONE DAY
PASSED THRO' No 4 A S T.M SIEVE
GRAVEL ANALYSIS IF PORTION RETAINED > 50%
GRAVEL SIZE FRACTIONS ENTERED IN FORM
DATA SHEET PREPARED
MATERIAL PASSING THOROUGHLY MIXED AND SAMPLED
SAMPLE PUT UP SEPARATELY TO P&S M.O
INSPECTED BY P&S.M.O FOR COLOUR & TEXTURE
FINES AND PLASTICITY JUDGED BY P&S.M.O BY 'FEEL'
RESULTS ENTERED BY P&S.M.O IN HIS PERSONAL REGISTER AND COMPARED
CHECKED AND PUT UP TO P&S.M.O
RETEST ORDERED WHERE NECESSARY
FORM B- PARTICULARS OF SAMPLES PREPARED
ABOUT 3000 GMS TAKEN AND OVEN DRIED
ABOUT 1500 GMS OF SAMPLE TAKEN
PASSED THRO' 40 MESH
200 GMS TAKEN FOR SIEVE ANALYSIS
SOAKED IN WATER OVER NIGHT
ELECTRICALLY STIRRED AND DISPERSED
WET SIEVED THROUGH 200 MESH
MATERIAL RETAINED OVEN DRIED
SIEVED THROUGH A S T M 4, 16, 30, 50, 100 & 200 MESHES
MATERIAL RETAINED IN EACH WEIGHED
50 OR 100 GMS TAKEN FOR HYDROMETER ANALYSIS
SOAKED IN WATER OVER NIGHT
ELECTRICALLY STIRRED AND DISPERSED
TRANSFERRED TO 1000 CC JAR & VOL MADE UP TO 1000 CC
AGITATED TO FORM UNIFORM SUSPENSION
HYDROMETER & THERMOMETER READINGS TAKEN AT STATED TIMES
CORRECTION FACTORS OBTAINED FROM CALIBRATION TABLES
1000 GMS TAKEN FOR COMPACTION & U.C.S TESTS
200 GMS TAKEN EACH TIME KNOWN QUANTITY OF WATER ADDED & MIXED
PUT IN ABBOTS CYLINDER AND COMPACTED WITH 55 STROKES
SOIL CAKE EJECTED OUT & LENGTH MEASURED
CAKE PLACED IN U.C.S APPARATUS AND LOADED TO FAILURE
FAILURE LOAD & DECREASE IN LENGTH NOTED
REPEATED FOR DIFFERENT M C IN STEPS OF 2%
200 GMS TAKEN FOR PERMEABILITY TEST
ONE ABBOTS CAKE AT O.M.C PREPARED
CAKE PLACED IN-SIDE PERMEAMETER
ANNULAR SPACE FILLED WITH WAX. CAKE TRIMMED TO CORRECT LENGTH
PERMEAMETER SET UP
TIME ALLOWED TO OBTAIN STEADY CONDITION
LEVELS IN STAND PIPES MEASURED AT DIFF. TIMES
800 GMS TAKEN FOR SHEAR TEST
FOUR ABBOTS CAKES AT OMC PREPARED
CAKE TRIMMED TO CORRECT LENGTH & ENCASED IN SHEAR-BOX
SHEAR BOX CLAMPED TO BASE PLATE AND CAKE SATURATED
FIXED IN LOADING MACHINE CLAMP REMOVED NORMAL LOAD APPLIED
ELECTRIC DRIVE STARTED-YIELD POINT READING NOTED
REPEATED FOR 4 DIFFERENT NORMAL LOADS
PORTION TAKEN WATER ADDED AND MIXED
SOIL PAT PLACED IN LIQUID LIMIT APPARATUS
STANDARD GROOVE MADE
DETERMINED No OF BLOWS TO JOIN
MOISTURE CONTENT DETERMINED
REPEATED FOR 4 MOISTURE CONTENTS
FLOW CURVE DRAWN AND LL READ OUT
SMALL QUANTITY MADE INTO A BALL FOR PLASTIC LIMIT TEST
BALL ROLLED ON PAPER PLACED ON TABLE
CONTINUED UNTIL SOIL THREAD BREAKS AT 1/8" DR
BROKEN THREADS PLACED IN METAL CANS & WEIGHED
OVEN DRIED COOLED & WEIGHED AND M.C DETERMINED
REPEATED TWICE
A SMALL QUANTITY TAKEN FOR F.M.E TEST
DROP OF WATER ADDED TO SMOOTHED SURFACE OF SAMPLE
IF DROP IS ABSORBED RAPIDLY M.C INCREASED SLIGHTLY
CONTINUED UNTIL DROP REMAINS UNABSORBED FOR 30 SECS.
M.C DETERMINED AT PLACE WHERE DROP IS ADDED
REPEATED TWICE
PORTION TAKEN EXCESS WATER ADDED & MIXED
SHRINKAGE DISH JUST FILLED
WEIGHED WITH WET PAT
AIR DRIED
OVEN DRIED
DRY PAT WEIGHED
VOLUME OF DRY PAT DETERMINED
CONSOLIDATED STATEMENT OF TEST RESULTS PREPARED
REPORT DRAFTED BY P&S.M.O
FAIR COPIED AND DESPATCHED
FOR DEPARTMENTAL SCHEMES TEST REPORT CONSISTS OF PARTICULARS OF SAMPLES DATA SHEETS AND CONSOLIDATED STATEMENT OF TEST RESULTS WITH AN INTERPRETATION OF TEST RESULTS AND RECOMMENDATIONS BY THE P&S.M.O
FOR EXTERNAL SCHEMES THE REPORT CONSISTS OF PARTICULARS OF SAMPLES DATA SHEETS CONSOLIDATED STATEMENT OF TEST RESULTS AND READINGS FOR INDIVIDUAL TESTS
RESULTS ENTERED IN APPROPRIATE STANDARD FORMS & CALCULATIONS CARRIED OUT

Fig. 1



● INDICATES TRIAL PIT FROM WHICH SOIL SAMPLES WERE RECEIVED AND ANALYSED.
RALLAPAD RESERVOIR DAM SITE
SOIL SURVEY MAP
SCALE 0 1 2 FURLONGS
GRAVELLY REGIONS ARE SHADED YELLOW
No. 9/52
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14
H
G
F
DL
A
B
C
D
E
RIVER
MANNERS
DAM LINE
ROAD
TO LINGA SAMUDRAM
PENTRALA
SERVICE ROAD TO CAMP SITE
NEKUNAMPET
ROAD
TO KALIGIRI
CHECKED BY:-
TRACED BY:-T·R
HEARTING SOIL
CASING SOIL
STIFF CLAY
14.3.52
PHYSICS & SOIL MECHANICS OFFICER

Fig. 2

ment, to mention only a few, have sought to take advantage of the facilities offered by the Laboratory and have sent their samples for tests here. In such cases the complete report of test results is sent without however including an interpretation of such results. A nominal fee is charged for the work done in accordance with a relevant Government order on the subject.

In the case of Departmental works, on the other hand, a full report is furnished containing the significance and import of the test results and the recommendations regarding the suitability or otherwise of the different samples towards constructional utility.

These test reports are found adequate for all practical purposes in the case of minor works like tank formation or restoration schemes, foundations for buildings etc. More detailed and comprehensive work is carried out, on the other hand, in the case of larger projects. First a preliminary soil survey for borrow areas in the neighbourhood of the dam line is made out. The standard procedure is to cover the area with a square grid of trial pits, usually at one furlong centres, and the soil strata exposed are tested, classified and mapped out. The soil survey map of the Rallapad Project reproduced as Fig. 2 will give an idea of the type of work herein envisaged. The areas where hearting and casing type of soils are available are marked out. A rough quantity survey is also made against construction demands. The leads to be allowed are worked out and indicated.

When the actual construction starts, it frequently happens that a more detailed soil survey map is required for drawing up a predetermined and specific programme of utilising the available borrow materials in the different reaches of the earth embankment under construction, with the emphasis placed on the ultimate economies in excavation and haulage. Trial pits at 200 ft. centres or nearer are put in during this phase of the work and the soils classified and indicated in a detailed soil map of the area. The type of this detailed soil map drawn for the Manimuthar Project is herewith reproduced in Fig. 3. About 500 trial pits have already been put in and some 500 more trial pits are still to be put in. The detailed map is only partly drawn and will be extended to other areas in the waterspread to the extent necessary. The areas where the stratum of the hearting type of soil overlies or underlies that of the casing type, the areas where sand or gravel strata are obtained and such other relevant features are all separately reported on. A complete picture of the soil potentialities in the area is hereby obtained and the authorities in charge of the construction can, with its help, accurately programme their work in order to ensure maximum economy in cost and in effort.

The Laboratory also co-operates with the construction authorities in order to afford full Laboratory control of the compaction effected in the earth dam or embankment. The greatest difficulty in such a control programme is to maintain optimum moisture of the soil layer being rolled in and compacted. The Laboratory determination of moisture contents takes time and is such as to delay and interfere with the progress of the work. A definite need existed for quite a long time, for devising a method of testing that will render superfluous the need of determining moisture contents while at the same time ensuring in effect the maintenance of the moisture in the soil at its optimum value so as to obtain the maximum compacted density. It is a matter on which the Laboratory can take justifiable pride that it has succeeded in developing such a method which in addition is so easy and simple that with the minimum training, any person of average intelligence can be educated to carry out the tests satisfactorily and with accuracy. The method is fully described in an article included in the Station's Annual Report for 1950. It consists in finding by a simple method the maximum realisable density on compaction from the borrow material, and later use the same method to objectively verify that the soil spread out for rolling has got the correct moisture content (at its optimum value) so as to enable the same maximum density to be realised on compaction. An undisturbed cylindrical sample of the rolled embankment is taken to determine the final realised density and to see whether it tallies with the maximum realisable density previously determined. The pieces of equipment necessary for carrying out these tests have all been designed and manufactured in the workshops attached to the Research Station.

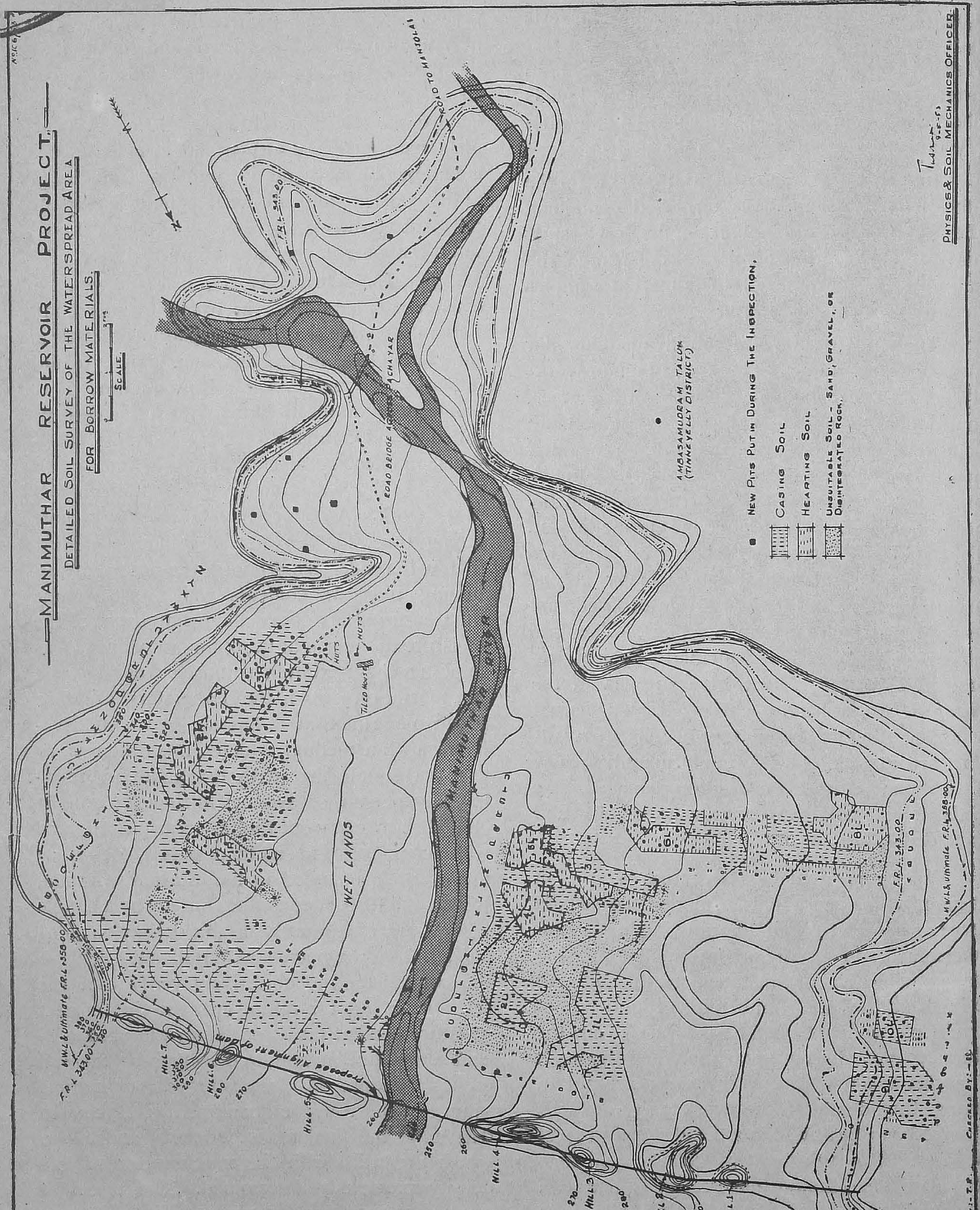

MANIMUTHAR RESERVOIR PROJECT.
DETAILED SOIL SURVEY OF THE WATERSPREAD AREA
FOR BORROW MATERIALS.
SCALE.
ROAD TO MANJOLAI
ROAD BRIDGE ACROSS PACHAYAR
AMBASAMUDRAM TALUK
(TINNEVELLY DISTRICT)
NEW PITS PUT IN DURING THE INSPECTION.
CASING SOIL
HEARTING SOIL
UNSUITABLE SOIL - SAND, GRAVEL, OR DISINTEGRATED ROCK.
WET LANDS
TILED HOUSE
HUTS
Proposed Alignment of Dam
HILL 1
HILL 2
HILL 3
HILL 4
HILL 5
HILL 6
HILL 7
PHYSICS & SOIL MECHANICS OFFICER

Fig. 3

Spare sets are always held in stock to be sent to the project sites whenever wanted. Dozens of such sets of equipment are being used in the different projects.

The results of these control experiments are recorded in suitably designed forms and copies of such daily reports are obtained in the Laboratory. These are checked up carefully and recorded in large scale charts which in addition to giving the compacted densities at different points in the Longitudinal section of the dam, also afford an ocular evidence within the laboratory, of the day to day growth of the embankment as the construction programme proceeds.

Moreover undisturbed samples from a number of predetermined points at regular intervals along the length of the dam and at points suitably staggered on verticals therein are required to be taken and sent to the Laboratory. All the significant tests are carried out on these undisturbed samples, including its bulk density, voids characteristics, compressive strength and sheer strength at the compacted density under saturation conditions. These results are recorded for future reference. In most cases undisturbed cylindrical samples of the compacted embankment at the predetermined points are taken out and preserved as representative of the materials used and compactions effected therein. A complete picture of the type of material used in the different sections and the compaction effected at the different points is thus gained and can be correlated later with reference to any normal or abnormal behaviour of the embankment at a future date.

In the case of major works, like, for instance the Manimuthar Project, a field Laboratory is organised at the project site itself. The work of the field Laboratory is to carry out the tests necessary for a detailed soil survey during construction and for the control of embankment compaction.

In the case of Malampuzha Project, a novel method was tried, instead of setting up a field Laboratory at site, as the earth work involved in the project headworks was not large enough to warrant such a laboratory to be established. Samples of soils to be used in construction were received daily at the Madras Research Station, tested immediately and reported on by return of post. The report contained instructions as to the suitability or otherwise of the samples sent. The samples were obtained a few days in advance of the time when they were to be laid out and rolled, and the test results reached the construction site in time to help in the proper choice of the soils to be used and in knowing beforehand the maximum density to be expected on compaction. Though initially a certain doubt was entertained as to the feasibility of this method of testing day to day samples from a distance of about 350 miles from the construction site, the method in actual practice was found to work out satisfactorily and to afford full laboratory facilities to the construction authorities without in the least interfering with or delaying their work.

The above account gives some idea of the intensely practical aid the Laboratory is giving to the construction work at the different centres. There is of course considerable theory involved as basis of the work carried out in the Laboratory: but, when affording help to the construction, the theory is kept well in the background and the practical aspect is stressed exclusively, and in such simple terms, that even the least technical worker can understand its scope and significance.

It will have to be mentioned further that the equipment used in the Laboratory and in the field have all been designed and manufactured in the workshops attached to the Laboratory using exclusively local talent and local materials. The Laboratory takes pride in recording that no single piece of equipment, as a unit, has been imported from abroad. Simplifying the apparatus wherever possible and making them locally in the Laboratory workshops have not only considerably reduced the capital cost of building up the Research Station but have also made it possible to programme for a rapid development, avoiding the usual bottlenecks and delays inherent in depending wholly or partly on foreign imports. This aspect of making locally all the necessary apparatus has been warmly commented upon by he many technical visitors to the Laboratory, "Nowhere have I seen so many locally made tnstruments and apparatus for doing various kinds of rapid tests with accuracy on a par with the best imported products" writes Sri S. K. Ghose, Dy. Director, Bihar Institute of Hydraulics and

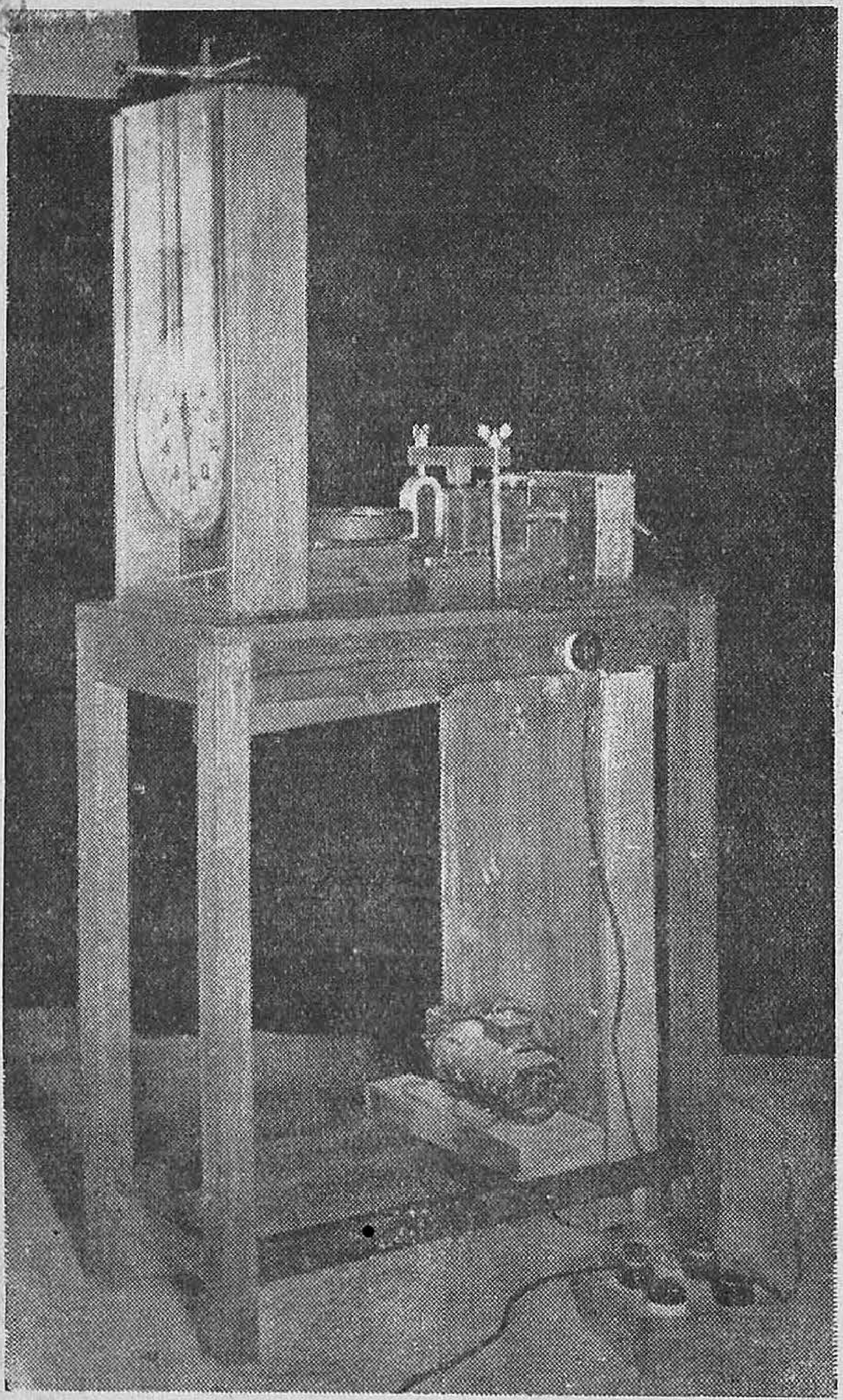


Figure 4 : Box shear apparatus with electric drive as designed and used in the Laboratory.

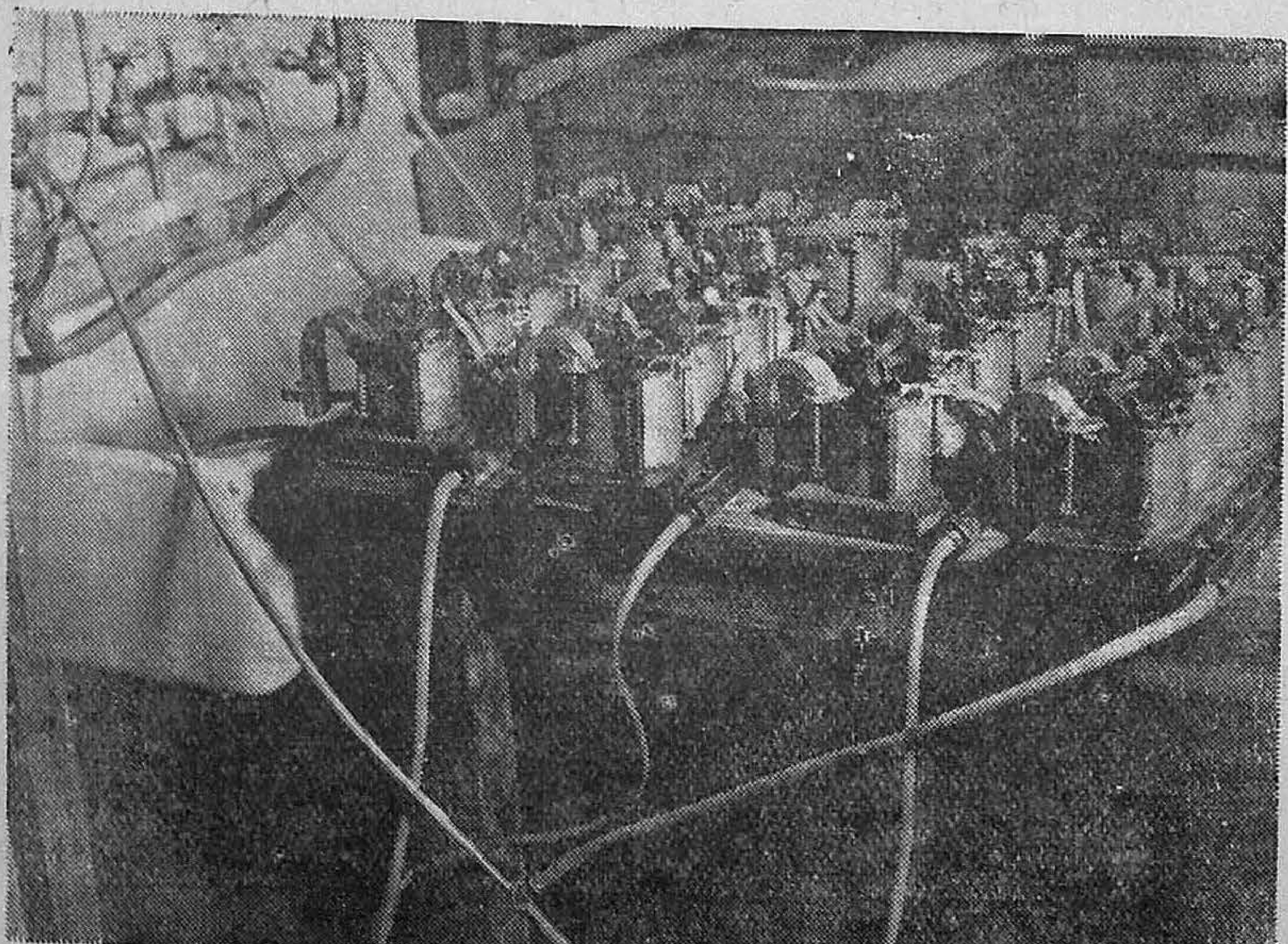


Figure 5 : A bank of cylindrical shear boxes in which the samples are getting saturated.

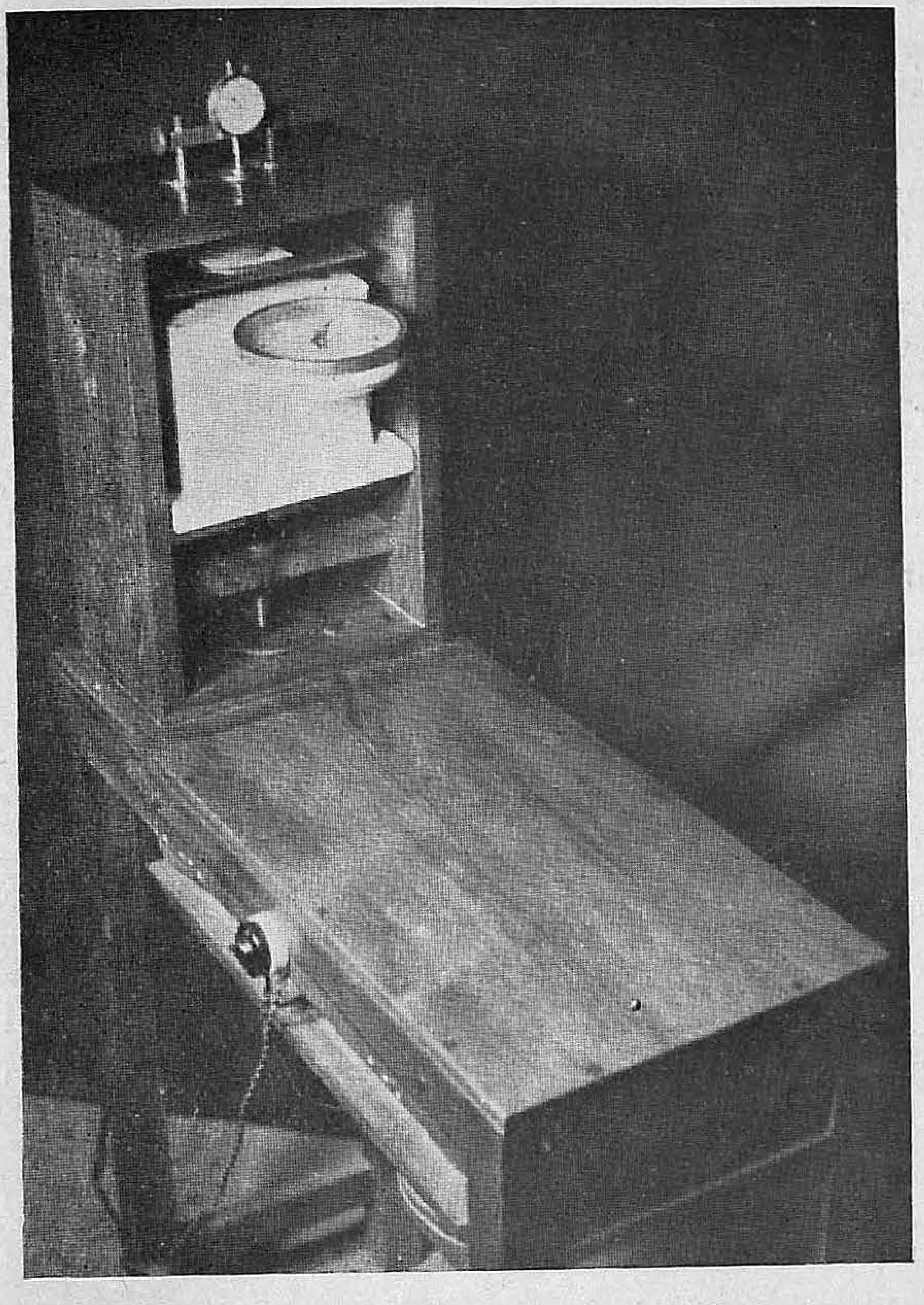


Figure 6 : The unconfined compression apparatus with eletric drive as built in the Laboratory.

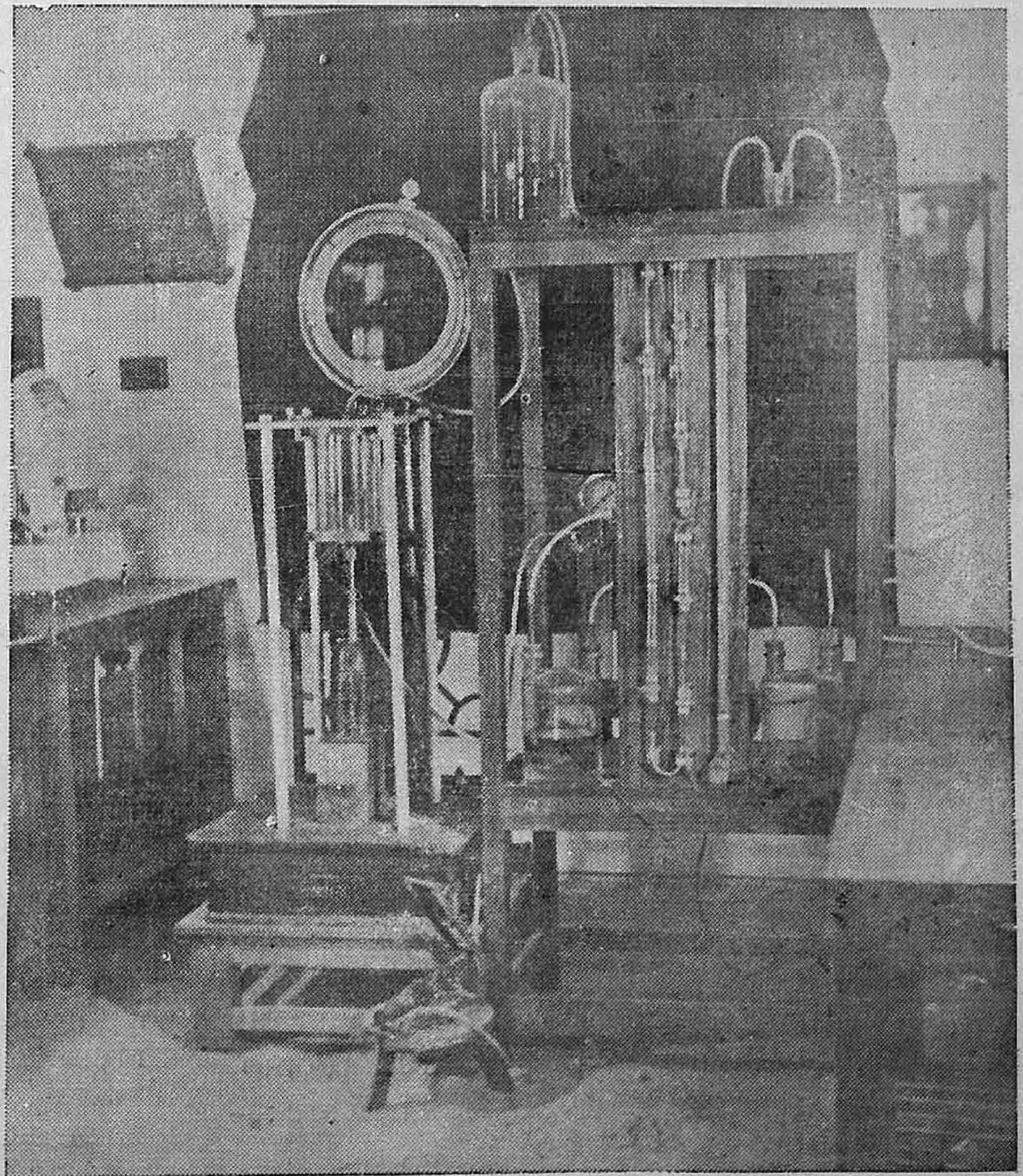


Figure 7 : The triaxial shear machine as designed and built in the Laboratory.

Allied Research. Mr. F. C. Rogers compliments authoritatively 'as a member of the committee D-18 American Society for Testing materials' that he can "express great satisfaction at the equipment, test procedures and activity of the Soil Engineering Research Station." Indeed the equipments, as developed and used in this Laboratory, have gained such wide approbation, that a large number of requests have been coming in recently from a number of institutions, for making and supplying such apparatus to them.

The Laboratory has also developed a Physics Section, a Chemistry Section and a Geology Section and has programmed for diverse fields of work like agronomical soil survey in ayacut areas, Geophysical methods of locating firm rock contours below construction sites, Photoelastic and other methods of stress analysis etc. It has also carried out basic research studies and significant results of fundamental importance have been obtained. But, an account of such activities has to be deferred for the present for lack of space. Only the work relating to engineering uses of soils is dealt with in this paper.

It may be pointed out finally that this Research Station is probably the earliest such Laboratory in the country and organised by one who had not even got the advantage of visiting foreign institutions of its kind. The research station on the other hand has been developed rapidly from start. As early as 1947, barely a year after the starting of the Laboratory, Sri. A. N. Khosla complimented it as "a remarkable achievement in so short a time".

The Laboratory moreover, from the beginning has served as a model for the later research stations of a similar kind that have sprung up in the different centres of the country—in Bombay, in West Bengal and in other places. "This information (regarding tests and apparatus) will be very useful to me in setting up a Central Laboratory at Poona" so wrote Shri D. V. Joglekar, Director, Central Waterways Irrigation Navigation Research Station, Poona. Mr. T. A Ferrant after visiting the Laboratory was "impressed with the initiative in developing original techniques" and trusted that his visit to this Laboratory "will be profitable to Australia". Mrs. B. Coldicott of the Department of Engineering, University of Melbourne was "inspired to see such initiative" and hoped she would be able "to convey some of this inspiration to the University Students" on her return.

The work of the Laboratory and the successes that it has registered are all due to the disinterested services and devoted co-operation of every one on the staff of the Laboratory. Shri Kanwar Sain, Chairman, Central Water and Power Commission complimented the "excellent Laboratory where workers take pride in their work with unusual and commendable zeal". In spite of great many disturbing factors, it is that enthusiasm for ones work and the joy of doing it well that sustain the workers in the Laboratory and still enable them to put forth abundantly of their best towards the good work this Research Station is continuing to do.

Ourselves

(*Continued from page 41*)

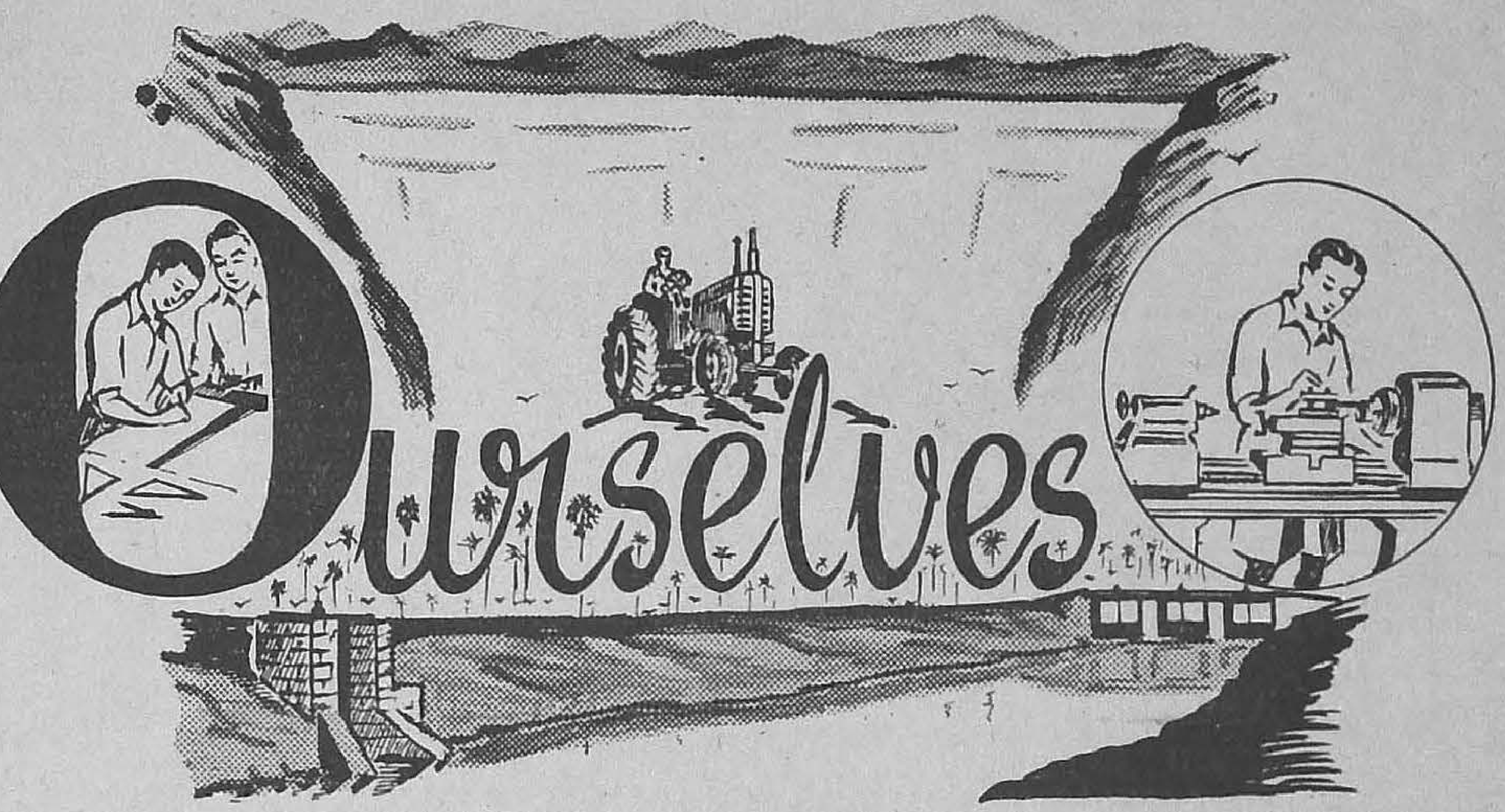
for having inherent faith in themselves, incessant devotion to duty and absolute integrity in their actions.

* * * *

Recently two of Senior Executive Engineers (Special Grade) of the department were promoted as Superintending Engineers. They are :—

(1) Sri R. Lakshmana Iyer, B.E.,
(2) Sri K. Harihara Iyer, B.A., B.E.

With the advent of the partition, our Engineers are going to divide themselves into two camps. The mutual goodwill and co-operation which has made the department's work outstanding in the past would continue as before and it is earnestly hoped that the contributions to the "Era" will be sent whenever possible by the Engineers in the sister State. These will be acknowledged and articles duly published in the journal.



This is a new feature that is being introduced in the "Era", from this issue onwards to recount the several phases of activities of "Ourselves", with particular reference to the personnel of the Department.

As this is the last issue of the journal before the partition, the first phase of the "Era" is concluding and it would be appropriate to recount about the ushering in of the magazine and how it has grown up.

The journal was inaugurated in January 1950 by the then Chief Engineer (Sri A. R. Venkatachari B.E., M.I.E. (Ind.), M.Am. Soc. C.E., I.S.E.) who in defining its aims stated, in the Editorial of the first issue as follows:

> "It is in keeping with the spirit of the times and advance of Engineering and Scientific knowledge that a magazine of this sort should be available for all workers in the field of irrigation to know the latest in Engineering in a practical form so as to be able to apply it in their daily tasks. From time to time will be published in this Magazine, articles on all matters connected with design and construction; the work done in our research stations, in the department, and also useful information collected from foreign publications."

The progress of the journal from its inception has been on satisfactory lines and there has been ample demand for it. For the first number, 324 copies were struck and the current figure is 600 copies. This phenomenal growth has been made possible by the co-operation of the staff in general and in particular the efforts of Sri M. R. Balasubramaniam, Ex Executive Engineer, of the Gaugings Division and Sri S. Ramachandran, Ex-Assistant Engineer, Technical (General), who had been entrusted with the task. The experience gained by Sri S. Ramachandran in this work has been sought to be made use of in the new post which he has since taken up with the Central Board of Irrigation (Publications Branch.)

It will also be appropriate in this context to name the services of Sri A. K. Raman, Junior Engineer who has made the journal take a proper and elegant shape by his minute scrutinies and minute by minute touching up.

* * * *

The Engineers of this Department were given valuable advice by the Chief Minister, Sri C. Rajagopalachari during the Annual Meeting of the Service Associations held at Madras on 12—7—'53 when he inaugurated the session. After paying a high tribute to the efficiency of the Engineers, he pointed out to them the necessity for preparation of economical estimates and execution of works on a cheap basis without sacrificing efficiency so as to utilise the limited available resources to the maximum advantage. He commended to the Engineers the necessity

(Continued on page 40)

Explanatory Notes for the representations on the facing page :—

The pictorial representations on top and the graphical representation at bottom in the facing page give details of "THE IMPORTANT RIVERS IN THE MADRAS PRESIDENCY AND THEIR AYACUTS."

The following explanatory notes are offered :—

Pictorial Representation at Top :—

Numbers in firm circles represent the ayacut under each river in Lakhs of acres.

Graphical Representation at Bottom :—

The Ayacut under each River in Lakhs of Acres is furnished.

The Ayacut under each of the rivers in Lakhs of acres is cumulatively added up in the following manner :—

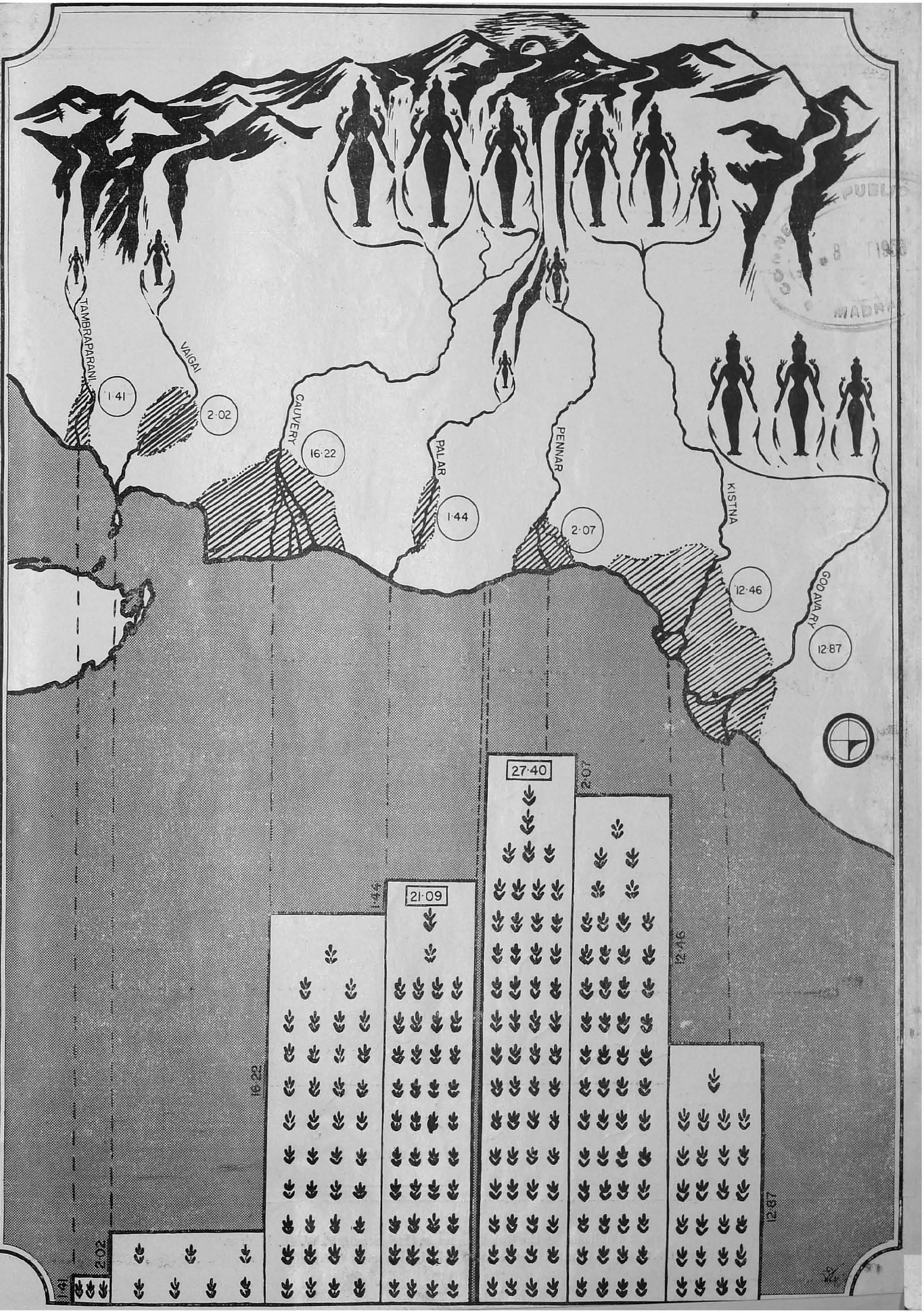
Andhra State — Right to Left
Residuary State — Left to Right

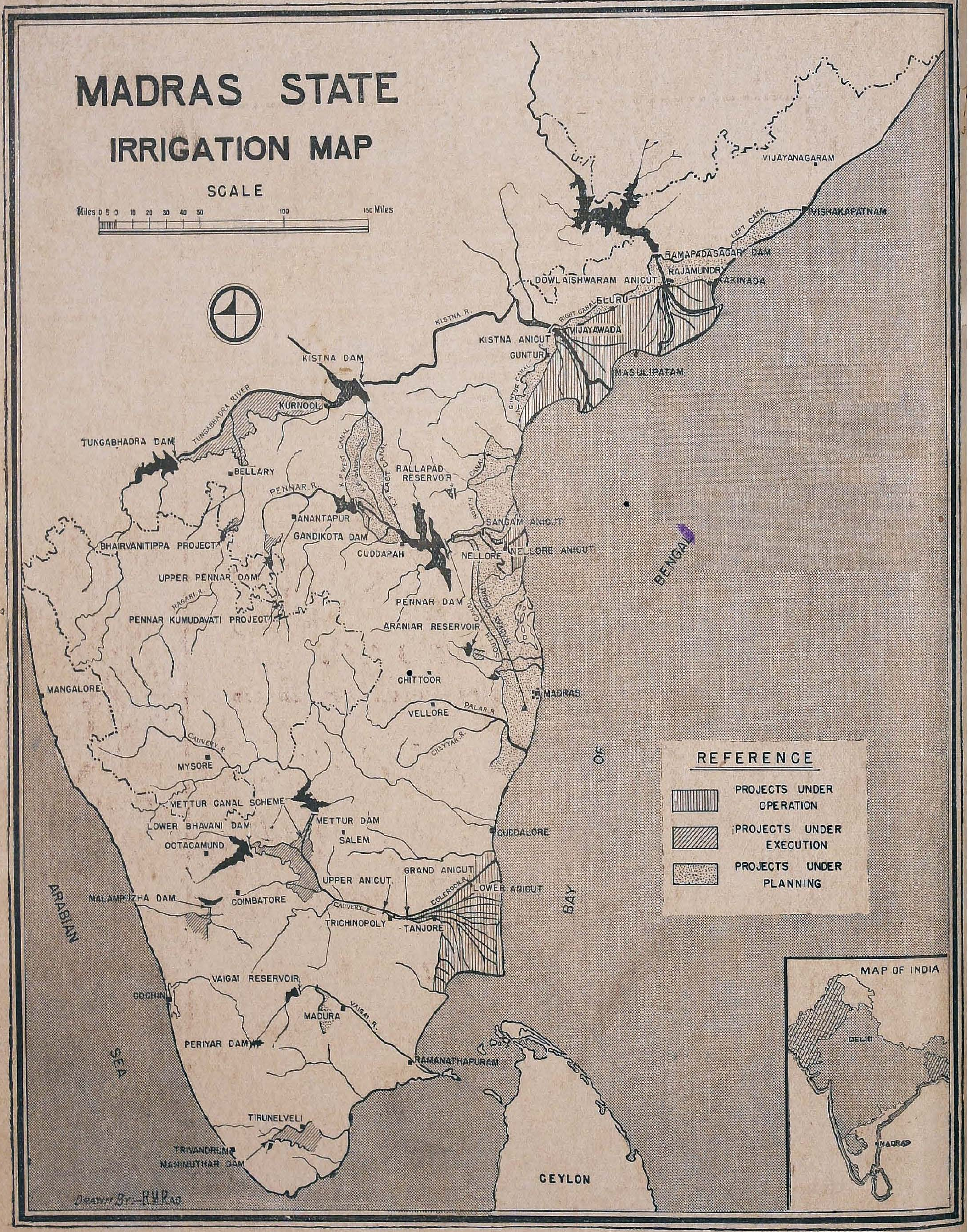
Numbers in Rectangles represent the total of ayacuts in Lakhs of acres under all the rivers in each of the States.

General :—

The total ayacut under River Cauvery includes 2.45 Lakhs against Cauvery-Mettur Project.

THE JAPANESE METHOD
OF PADDY Cultivation
RAISED SEED BED
SELECTED SEEDS
LINE SOWING
INTER-CULTURE


TAMBRAPARANI
VAIGAI
1·41
2·02
CAUVERY
16·22
PALAR
1·44
PENNAR
2·07
KISTNA
12·46
GODAVARY
12·87
21·09
27·40



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